R-05-25

Study of tunnelling through water-bearing fracture zones

Baseline study on technical issues with NE-1 as reference

Yanting Chang, Robert Swindell, Ingvar Bogdanoff,
Beatrice Lindström, Jens Termén
WSP Sweden

Peter Starsec, SGI Sweden

April 2005

Svensk Kärnbränslehantering AB
Swedish Nuclear Fuel
and Waste Management Co
Box 5864
SE-102 40 Stockholm Sweden
Tel 08-459 84 00
+46 8 459 84 00
Fax 08-661 57 19
+46 8 661 57 19

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Preface

This report presents results from a baseline study concerning technical issues related to tunnelling through water-bearing fracture zones. SKB aims with this study to improve the generic knowledge of tunnelling through such zones at different depths. As a reference was the experiences of tunnelling through a water-bearing fracture zone with the access ramp to the Äspö Hard Rock Laboratory in Sweden chosen. The content of the report shall be used as a reference for specific issues in the rock engineering design of a deep repository facility.

The study was started in the spring of 2004 and has been carried out by WSP Sverige AB. The team leader has been Yanting Chang. The following experts have also made efforts in the study: Lars Hässler, Johan Ingvarson, Sven Jonasson and Jacek Jakubowski. The project was divided in five stages. For each stage a verbal presentation was made to the reference group and a working document was compiled and reviewed.

On behalf of SKB, Eva Widing was responsible for the project. Mats Holmberg (Tunnel Engineering AB) has been the project coordinator and reviewing the study together with a reference group consisting of Rolf Christiansson (SKB), Håkan Stille (KTH) and Lars Olsson (Geostatistik). The draft to the final report was presented in November 2004 and valuable comments and proposals for improvements were received from the external reviewers John A Hudson, C Derek Martin and Håkan Stille.

Stockholm, April 2005

Eva Widing/

Rolf Christiansson

Summary

The Swedish Nuclear Fuel and Waste Management Co (Svensk Kärnbränslehantering AB, SKB) is responsible for the management of Sweden's nuclear waste. SKB is investigating various designs for the construction of an underground deep repository for spent nuclear fuel at 500 m–600 m depths. For the construction of an access tunnel for such a deep repository, the possibility of encountering a water-bearing fracture zone cannot be discounted. Such a zone named NE-1 (deformation zone in accordance to SKB's terminology) was encountered during the construction of the Äspö Hard Rock Laboratory (HRL) and difficulties with large water inflows were reported.

With the aim to assess the feasibility of different technical solutions, SKB commissioned a baseline study into the passage of an access tunnel through a water-bearing fracture zone at three different depths (200 m, 400 m and 600 m). The objectives of this baseline study are to:

- Increase the knowledge of possible technical solutions for tunnelling through water-bearing fractures zones with the characteristics of the brittle deformation zone NE-1 at different depths, namely 200, 400 and 600 metres;
- Form a reference document to assist the engineering design and construction work for the passage through such a water-bearing fracture zone;
- To highlight the engineering parameters that should be obtained to facilitate design for the passage through water-bearing fracture zones.

The study has been carried out in the following five stages:

A. Compilation of the relevant data for deformation zone NE-1;

B. Problem identification and proposal of technical solutions;

C. Identification of hazards to be involved in the tunnel excavation;

D. Recommendations and conclusions for further investigations;

E. Documentation of the results in a final report.

The analyses will be expressed in statistical/probabilistic terms where appropriate.

In order to specify the precondition that will be valid for this study, a descriptive model of the water-bearing fracture zone is established, based on the review of the geological and hydrogeological characteristics of the deformation zone NE-1. In the descriptive model, the water-bearing fracture zone consists of an 8 metre wide central core zone and a 15 metre wide transition zone sited on either side of the core zone. Rock mechanical and hydrogeological properties of the rock mass as well as *in situ* rock stresses are assigned in the descriptive model.

To highlight the important technical issues in tunnelling through water-bearing fracture zones, system analysis and problem identification based on a literature review of relevant case histories are conducted. The identified important technical issues, namely large water inflow and tunnel stability, will be the objects to be analysed in this study.

Control of water inflows is the key issue for the safe passage of a tunnel through a water-bearing fracture zone with the characteristics of NE-1. Technical issues associated with the two most used methods for water inflow control, namely grouting and ground freezing are

discussed. The analyses regarding water inflows associated with grouting are presented. The degree of difficulty for water inflow control increases with depth. The study indicates that control of water inflows at all the depths could be achieved by grouting with current technology. But ground freezing might be an alternative for the core zone, for instance at a depth of 600 metres. Due to the high water pressure that may be encountered at a depth of 600 metres, precautions must be taken in the decision making process in selecting the most appropriate methods of groundwater control.

The deformation analysis indicates that large deformations are unlikely to occur in the transition zone, even at a depth of 600 metres. The reduction in rock mass quality in the core zone, however, is likely to result in large deformations at great depths. The estimated mean values of deformation for an unsupported tunnel in the core zone are 60 mm and 130 mm at depths of 400 and 600 metres respectively. The stability problems associated with such deformations can essentially be dealt with using conventional support methods in the form of rockbolts and fibre-reinforced shotcrete, however, the introduction of steel sets and the use of forepole umbrellas may be necessary.

Tunnel face stability is of crucial importance for the safety of tunnel excavations under high water pressure. For instance, face instability in form of flowing ground would result in catastrophic consequences, e.g. flooding of the whole tunnel, which may jeopardise the whole tunnel project. Nevertheless, such risks can be decreased considerably by careful probing ahead of the tunnel face. Another effective measure for preventing the face from collapse due to high water pressures is observation of the face. Large water leaking from fractures at the face is an early indication of the presence of water behind the face. Examination of the tunnel face for water seeping from fractures should therefore be emphasized during tunnelling operations.

Drilling and grouting in fractured rock under high water pressure could be troublesome. The use of the “blow-out preventer”, standpipes and T-valves could overcome the problems associated with drilling and grouting under high water pressure. However, it is advisable that the drilling work of such kind is contracted to specialist contractors.

Based on the analysis in this study, recommendations for further investigations with the aim of tunnel design are given. The study given in this report concludes that the control of water inflows is the key issue for the passage of a water-bearing fracture zone with the characteristics of NE-1 at great depths. The technical issues associated with grouting in fractured rocks under high water pressure should be further investigated in the coming stages of SKB’s design process.

Sammanfattning

Svensk Kärnbränslehantering AB (SKB) ansvarar för hanteringen av den svenska industrins använda kärnbränsle. SKB undersöker olika förslag till utformning av en underjordisk förvaring av använt kärnbränsle på djupet 500–600 m. Sannolikheten att träffa på en vattenförande sprickzon under konstruktionen av en tillfartstunnel till denna djupliggande förvaring bör inte ignoreras. En sådan zon betecknad NE-1 (deformationszon enligt SKB:s terminologi) påträffades vid byggandet av Äspö Hard Rock Laboratory (HRL), där svårigheter med stora mängder vatteninströmningar förekom.

Med målet att undersöka genomförbarheten av olika tekniska lösningar, har SKB låtit utföra en preliminär studie av en tillfartstunnel som passerar genom en vattenförande sprickzon vid tre olika djup (200 m, 400 m och 600 m). Syftet med denna preliminära studie är att:

- Öka kunskapen om rimliga tekniska lösningar vid passage av vattenförande sprickzoner med liknande egenskaper som deformationszon NE-1 vid den fortsatta projekteringen eller under genomförandet. Studien innefattar passage av en sprickzon på tre olika djup 200 m, 400 m och 600 m;
- Resultatet skall kunna användas som en referens för framtida arbete med projektering och under genomförandet av passager genom vattenförande sprickzoner av liknande karaktär;
- Ge rekommendationer för vilka undersökningar som är värdefulla för andra liknande problem.

Studien har utförts i följande fem steg:

A. Sammanställning av relevant data för deformationszon NE-1;

B. Identifiering av problem och förslag till tekniska lösningar;

C. Riskbedömning av de identifierade tekniska problemen;

D. Rekommendationer och slutsatser för fortsatta undersökningar;

E. Dokumentation av resultaten i en slutlig rapport.

För analyserna i denna studie har statistik- eller sannolikhetsbaserade metoder använts där det ansetts lämpligt.

För att bestämma de förhållanden som ska gälla för studien, upprättades en deskriptiv modell av den vattenförande sprickzonen, baserat på en genomgång av geologiska och hydrogeologiska egenskaper vid deformationszonen NE-1.

I den deskriptiva modellen består den vattenförande sprickzonen av en 8 meters bred central kärna och 15 meter breda övergångszoner placerade på vardera sidan av kärnan. Bergtekniska och hydrogeologiska egenskaper av bergmassan samt initiala bergspänningar är inlagda i den deskriptiva modellen.

För att belysa de tekniska problem som kan påträffas under tunnelbyggande genom en vattenförande sprickzon, har systemanalyser och problemidentifiering utförts baserad på en litteraturstudie av relevanta fall. De identifierade viktiga tekniska problemen stor vatteninströmning och tunnelstabilitet har analyserats i denna studie.

Kontroll av vatteninläckage är huvudproblemet för säkerställandet av passage genom en vattenförande sprickszon med egenskaperna liknande NE-1. Tekniska frågeställningar för de två vanligaste metoderna för kontroll av vatteninläckaget, nämligen injektering och frysning, diskuteras. Analyserna för vatteninläckaget vid användning av injektering presenteras i rapporten. Svårighetsgraden med vatteninläckaget ökar med djupet. Denna studie indikerar att vatteninströmning kan kontrolleras med hjälp av injektering på alla djup. Medan frysning kan vara användbart för kärnan av sprickzonen, t ex vid 600 meters djup. På grund av det extremt höga vattentrycket på 600 meters djup, måste valet av den lämpligaste metoden för hantering av vatteninläckage övervägas noga.

Stabilitetsanalyser indikerar att det finns ringa risk för stabilitetsproblem i övergångszonen på grund av stora deformationer. I kärnan av zonen är riskerna för instabilitet högre på större djup. Bergförstärkningen med bergbultar kombinerat med stålfiberarmerad eller stålnätarmerad sprutbetong bedöms vara tillräckligt för att säkerställa stabiliteten. Användningen av stålbågar eller ”spiling” kan dock vara nödvändig.

Tunnelfronten stabilitet är avgörande för säkerheten av tunneldrivning under höga vattentryck. Stabilitetsproblem som resulterar i s k ”flowing ground” kunna medföra katastrofala följder såsom översvämning i hela tunneln, vilket skulle äventyra hela projektet. Sådana risker kan dock minskas ansenligt genom att kontinuerligt utföra sonderingsborrningar framför tunnelfronten. Observationer av vatteninflödet från tunnelfronten är en annan viktig åtgärd för att minska risken för instabilitet.

Borrning och injektering i sprickigt berg under högt vattentryck är komplicerat. Användning av s k ”blow-out preventer” och ”standpipes” som säkerhetsåtgärder syftar till att kontrollera riskerna och skapar en säkrare arbetsmiljö. För speciellt komplicerade borrningsarbeten under svåra förhållandena rekommenderas att specialiserade entreprenörer anlitas.

Rekommendationer för fortsatta utredningar om tunneldrivning genom en vattenförande sprickzon är baserade på resultaten av denna studie. Studien kan sammanfattas med att kontroll av vatteninströmningar är en nyckelfråga vid passage av en vattenförande sprickzon med karaktärsdrag liknande NE-1 på stora djup. För SKB:s fortsatta projekteringsarbete rekommenderas en vidare detaljutredning av injektering i sprickigt berg under högt vattentryck.

Contents

1 Introduction

1.1 Background

The Swedish Nuclear Fuel and Waste Management Co (Svensk Kärnbränslehantering AB, SKB) is responsible for the management of Sweden's nuclear waste. SKB is currently in the process of investigating various designs for the construction of an underground deep repository for spent nuclear fuel. The preliminary layout consists of an access ramp, shafts, central areas and deposit areas (see Figure 1-1).

The planned deep repository is designed for a capacity of approximately 9,000 tonnes (4,500 canisters) of spent nuclear fuel. The underground portion of the repository is expected to extend over an area of 2–4 km^2 and will be located at a depth of between 400 and 700 m in crystalline rock.

SKB has conducted investigations at several sites to assess their suitability for construction of such a deep repository. Two sites, Oskarshamn and Forsmark, have been selected as potentially suitable and are being subjected to further investigation.

Site investigation results indicate that there may be several deformation zones within the chosen site areas. The construction of the access tunnel for the Äspö Hard Rock Laboratory (HRL), which was built close to the Simpevarp peninsula (see Figure 1-2) to conduct research in an undisturbed rock environment, encountered several brittle deformation zones (see Figure 1-2). One of these was the deformation zone named as NE-1 (see Figure 1-3). The passage of the deformation zone NE-1 proved to be particularly difficult and time-consuming, due primarily to high water pressures that resulted in large water inflows.

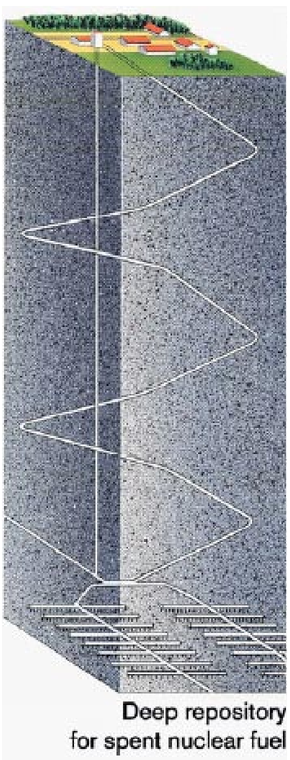


Figure 1-1. *Sketch illustrating a deep repository for spent nuclear fuel /SKB, 2003/.*

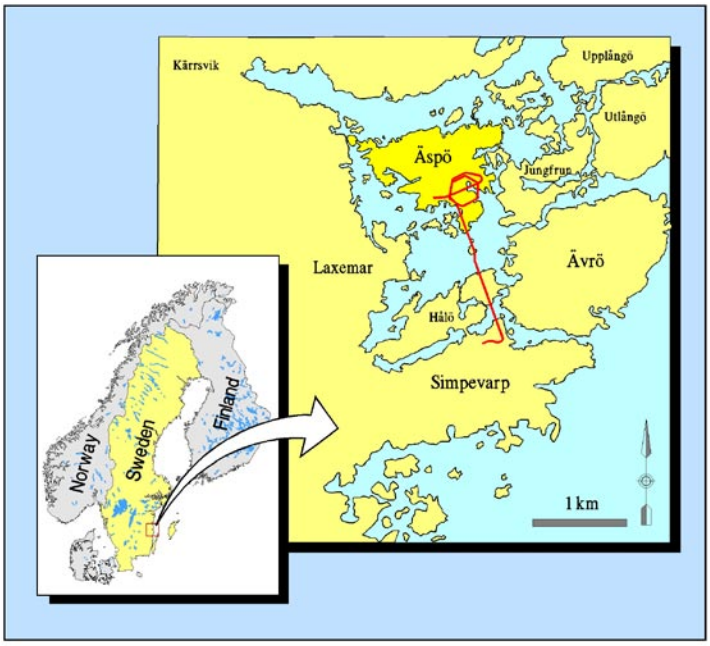


Figure 1-2. *Location of the Simpevarp peninsula and Äspö HRL /Rhén et al. 1997a/.*

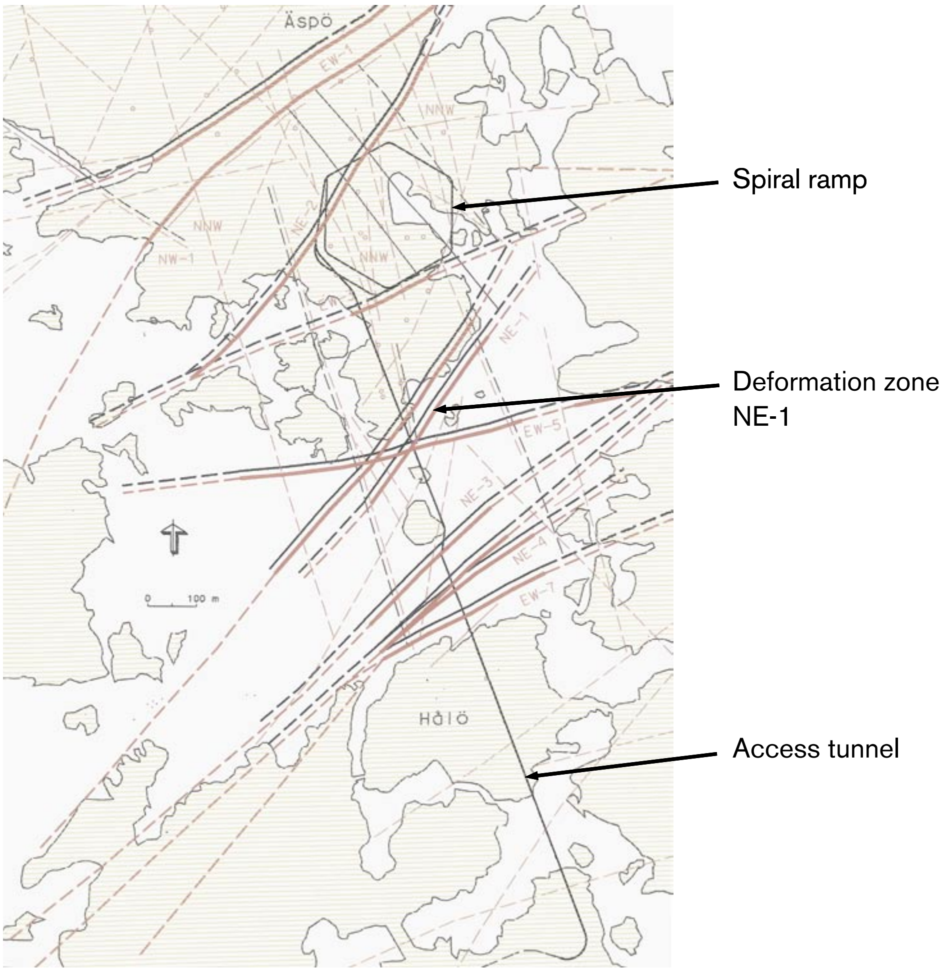


Figure 1-3. *Location of deformation zones and ramp system in the Äspö area /Stille et al. 1993b/.*

The possibility of encountering a brittle deformation zone during the construction of an access tunnel for a deep repository cannot be discounted. As a result, SKB commissioned a baseline study into the feasibility of different technical solutions for the passage of water-bearing fracture zones at three different depths (200 m, 400 m and 600 m).

1.2 Objectives of the study

The objectives of this baseline study are to:

- Increase the knowledge of possible technical solutions for tunnelling through water-bearing fractures zones with the characteristics of the brittle deformation zone NE-1 at different depths, namely 200 m, 400 m and 600 m;
- Form a reference document to assist the engineering design and construction work for the passage of such a water-bearing fracture zone;
- To highlight the engineering parameters that should be obtained to facilitate design for passage through water-bearing fracture zones.

The study has been carried out in the following five stages:

A. Compilation of the relevant data for deformation zone NE-1;

B. Problem identification and proposal of technical solutions;

C. Identification of hazards to be involved in the tunnel excavation;

D. Recommendations and conclusions for further investigations;

E. Documentation of the results in a final report.

It is worth noting that this study has not been aimed to provide detailed solutions to all the problems that would be encountered in tunnelling through a water-bearing fracture zone at such great depths. Recommendations of the efforts that will be needed in the coming stages of SKB's design process are therefore given in this report.

1.3 Project constraints

With the aim to provide a general description of the engineering issues associated with tunnelling through water-bearing fracture zones, the following project constraints and assumptions have been made:

- Design issues in terms of technical requirements, specifications and drawings are not included;
- Contractual and organizational issues have not been considered;
- The descriptive model based on the characteristics of deformation zone NE-1 has been used as the reference for the tunnelling conditions;
- The study is to be carried out for a 7×7 m, type B access tunnel (cross sectional area 45 m^2) in accordance with Layout E /SKB, 2002b/.
- The excavation method is assumed to be drill and blast. The grout medium has been restricted to cement grout;
- The analyses will be expressed in statistical/probabilistic terms where appropriate.

1.4 Structure of the report

The structure of the report is illustrated in Figure 1-4.

A descriptive model of the water-bearing fracture zone to be used in this study is given in Chapter 2. This model is essentially based on the review of the geological and hydrogeological characteristics associated with deformation zone NE-1. A full presentation of the outcome of the review conducted on the source material supplied for this study is given in Appendix 1.

System analysis and problem identification based on the passage of NE-1, together with experiences from relevant case histories is presented in Chapter 3. Chapter 4 comprises analyses relating to water inflow and possible solutions to restrict the degree of inflow, namely grouting and ground freezing are discussed.

Chapter 5 includes stability studies for over-stressed ground, face stability and rock block/wedge stability and Chapter 6 addresses the construction issues associated with tunnelling under high water pressures.

Hazard assessment for various scenarios are presented in Chapter 7 and excavation strategies are discussed in Chapter 8. Conclusions and recommendations from this study are presented in Chapter 9.

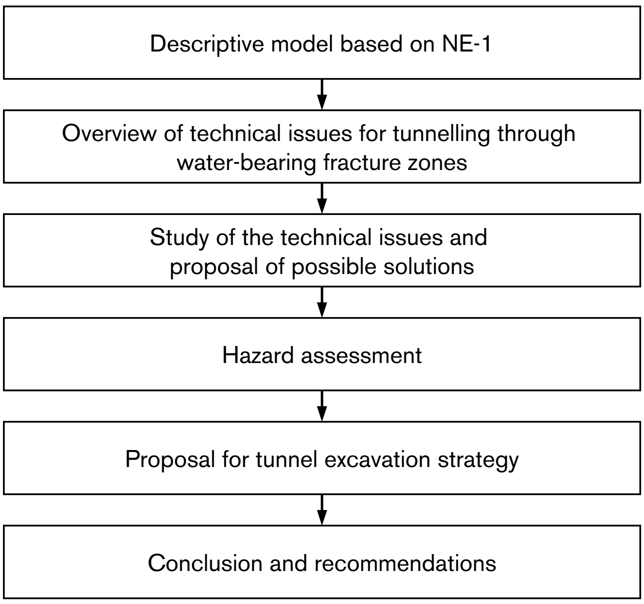


Figure 1-4. *Report structure.*

2 Descriptive model

2.1 General

The objective of this chapter is to provide a descriptive model for the water-bearing fracture zone at 3 different depths, 200, 400 and 600 m. The characteristics of deformation zone NE-1 are used as the basis for the descriptive model. A review of the data obtained from the passage of deformation zone NE-1 is given in Appendix 1. The descriptive model to be used in this study is an interpretation of the data of NE-1. The model includes the following parts:

- Engineering geological features.
- Hydrogeological features.
- Rock mass properties and *in situ* rock stresses.

2.2 Engineering geological features

The geological features of the descriptive model consist of an 8 m wide core zone of highly fractured and tectonized granite and mylonite. Within this core zone there is a 1 m wide section of clay gouge. A 15 m long transition zone, consisting of fractured fine-grained granite and diorite, is situated on either side of the 8 m wide core zone. A schematic sketch section displaying the geological features of the deformation zone is shown in Figure 2-1.

The fracture zone is assumed to dip 70° in the direction of the tunnel drive and the tunnel is to be driven perpendicular to the fracture zone. Eight non-water-bearing fracture sets and three water-bearing sets are included in the descriptive model (see Table 2-1).

The geological features are assumed to be applicable for all three depths (200 m, 400 m and 600 m) for this study.

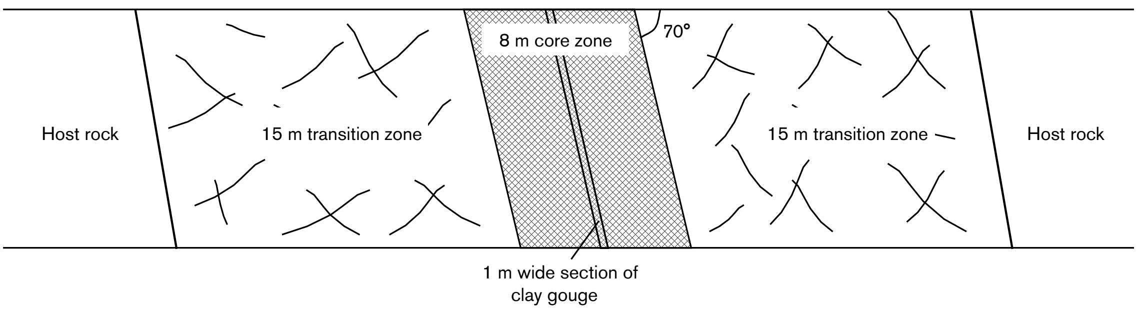


Figure 2-1. *Schematic section of the geological features in the descriptive model to be used for this study.*

Table 2-1. Fracture sets used in the descriptive model.

Waterbearing			Non-waterbearing		
Fracture set	Strike	Dip	Fracture set	Strike	Dip
JW1	230 (NE-SW)	35° NW	J1	284 (E-W)	Vertical
JW2	341 (NNW-SSE)	45° ENE	J2	225 (NE-SW)	50° NW
JW3 clay core	060 (NE-SW)	60° N	J3	045 (NE-SW)	30° SE
			J4	050 (NE-SW)	60° SE
			J5	094 (E-W)	60° S
			J6	120 (WNW-ESE)	35° SSW
			J7	310 (NW-SE)	38° NE
			J8	310 (NW-SE)	75° NE

2.3 Hydrogeological features

Major difficulties experienced during the passage of deformation zone NE-1 were due to large water inflows. One of the major aims of this study is to provide an estimate of water inflows associated with grouting for the passage of such a fracture zone. The hydrogeological properties and mechanics of water flow within a deformation zone such as NE-1 are complex. For the purposes of this study the rock mass will be treated as a homogeneous medium with respect to its hydrogeological behaviour. The key parameter for estimating the water inflows is hence the hydraulic conductivity (K) of the rock mass. All hydrogeological data obtained for the passage of deformation zone NE-1 in SKB's reports is however presented as transmissivity (T). Therefore, the hydraulic conductivity must be derived from the transmissivity. The following method for this transformation is proposed.

Assuming that a fracture zone represents a homogenous and porous media that behaves according to Darcy's law, and that the water pressure conditions in the media are uniform along a borehole, the following relationship between water inflow and transmissivity can be established:

$$\frac{Q_i}{\sum Q_i} = \frac{T_i}{\sum T_i} \qquad (2\text{-}1)$$

Where:

Q_i is the flow in a specific section of the borehole,

T_i is the transmissivity of the specific section,

$\Sigma\ Q_i$ is the total flow from the borehole,

$\Sigma\ T_i$ is the total transmissivity of the borehole.

The transmissivity of a borehole section T_i can then be calculated if the total inflow $\Sigma\ Q_i$ to the borehole, the total transmissivity $\Sigma\ T_i$ of the borehole and the inflow Q_i over the specific borehole section are known. The average hydraulic conductivity K_i over a specific section in a borehole is then, by definition, obtained as:

$$K_i = T_i/b_i \qquad (2\text{-}2)$$

Where:

K_i is the hydraulic conductivity of the specific section;

b_i is the length of the specific section.

Combination of equations 2-1 and 2-2 gives the following relationship:

$$K_i = \frac{1}{b_i} \cdot \frac{Q_i}{\sum Q_i} \cdot \sum T_i \qquad (2\text{-}3)$$

Using equation 2-3, the average hydraulic conductivity K_i over a specific section with length b_i can be obtained when the total transmissivity ΣT_i of the borehole, total inflow ΣQ_i from the borehole and the inflow Q_i over the specific section are given.

Based on the transmissivity data given in Table A1-14 and A1-17 in Appendix 1, hydraulic conductivities (K_i) have been calculated for both the core zone as well as the transition zone of NE-1 by using equation (2-3). Table 2-2 presents the details of the calculations, while Table 2-3 and Table 2-4 present the summarized results for the transition zone and the core zone respectively. The mean values presented in the tables are arithmetical means. The interpreted boundaries for the core and transition zone used for the calculation of the hydraulic conductivities are shown in Figure 2-2.

Table 2-2. Estimated hydraulic conductivity values based on measured transmissivities of NE-1.

Borehole no	Total transmissivity (m^2/s), ΣT	Total inflow (l/min), ΣQ	Length of specific section (m), bi	Inflow of specific section (l/min), Qi	Conductivity of specific section (m/s), Ki	Location in the fracture zone
HA1272A	4.40E-04	150	29	100	1.01E-05	Transition zone
			3	50	4.89E-05	Transition zone
HA1273A	4.90E-04	1,380	14	100	2.54E-06	Transition zone
			3	300	3.55E-05	Transition zone
			3	100	1.18E-05	Core of fracture zone
			3	880	1.04E-04	Core of fracture zone
HA1274A	4.40E-04	2,000	16	2,000	2.75E-05	Transition zone
HA1275A	4.70E-04	1,600	26	250	2.82E-06	Transition zone
			3	750	7.34E-05	Core of fracture zone
			0.8	600	2.20E-04	Core of fracture zone
HA1276A	4.90E-04	480	22	380	1.76E-05	Transition zone
			2.5	100	4.08E-05	Core of fracture zone
HA1282B	4.40E-04	27.6	10	20	3.19E-05	Outside of fracture zone
			13	4	4.91E-06	Outside of fracture zone
			8.5	3.6	6.75E-06	Outside of fracture zone
HA1283B	4.20E-04	1,350	31	200	2.01E-06	Transition zone
			4.5	1,150	7.95E-05	Core of fracture zone
HA1286B	4.70E-04	33	16	10	8.90E-06	Outside of fracture zone
			6	10	2.37E-05	Outside of fracture zone
			12	10	1.19E-05	Transition zone

Table 2-3. Estimated hydraulic conductivities for the transition zone of NE-1.

Test borehole	Hydraulic conductivity K (m/s)
HA1272A	1.01E–05
HA1272A	4.89E–05
HA1273A	2.54E–06
HA1273A	3.55E–05
HA1274A	2.75E–05
HA1275A	2.82E–06
HA1276A	1.76E–05
HA1283B	2.01E–06
HA1286B	1.19E–05
Min	2.0E–06
Mean	1.8E–05
Max	4.9E–05

Table 2-4. Estimated hydraulic conductivities for the core zone of NE-1.

Test borehole	Hydraulic conductivity K (m/s)
HA1273A	1.18E–05
HA1273A	1.04E–04
HA1275A	7.34E–05
HA1275A	2.20E–04
HA1276A	4.08E–05
HA1283A	7.95E–05
Min	1.2E–05
Mean	8.8E–05
Max	2.2E–04

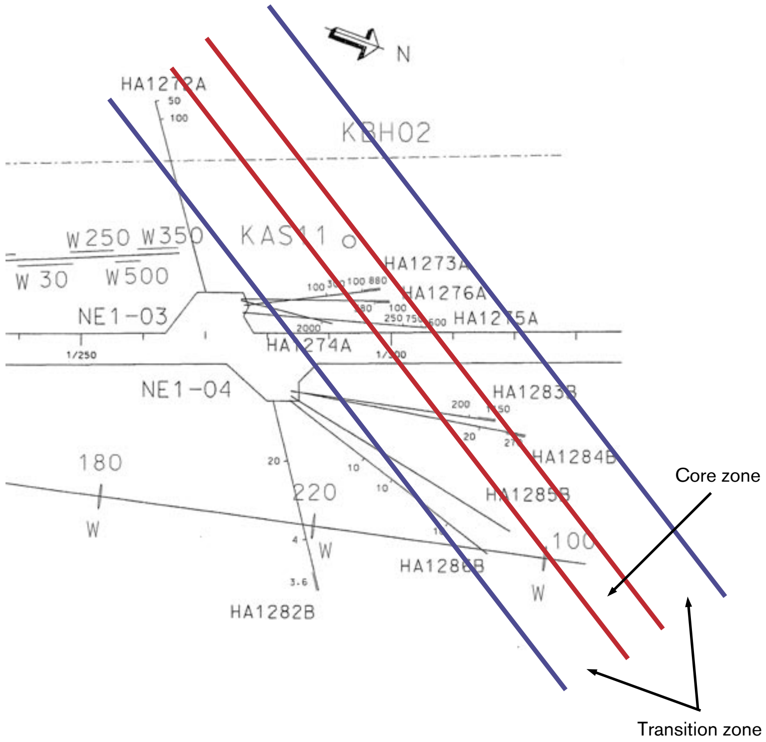


Figure 2-2. *Approximate division of the transition and core zone of NE-1 used for estimation of the hydraulic conductivities.*

The hydraulic conductivities quoted in Table 2-3 and Table 2-4 are based on data from the passage of deformation zone NE-1 at a depth of approximately 200 m. In theory it could be argued that an increase in depth (and hence *in situ* stress) should result in a decrease in hydraulic conductivity due to the closure of fractures. However, the data reviewed for this study suggests that the correlation between hydraulic conductivity and depth is weak (see Appendix 1). For the purposes of this study, it is assumed that hydraulic conductivity does not vary with depth. As a result, the hydraulic conductivity values presented in Table 2-5 are assumed to be representative for all three depths (200 m, 400 m and 600 m).

Table 2-5. Hydraulic conductivities proposed for the descriptive model.

	Hydraulic conductivity K (m/s)	
	Transition zone	Core of fracture zone
Min	2.0E–06	1.2E–05
Mean	1.8E–05	8.8E–05
Max	4.9E–05	2.2E–04

2.4 *In situ* rock stresses

Rock stress measurements were conducted in the area of the Äspö-HRL and stress relationships were developed for the Simpevarp site descriptive model /SKB, 2002a/. The results of these tests are summarised in Table A1-13 in Appendix 1. The stress measurement program was essentially conducted in competent rock, however, it is well known that the magnitude and orientation of stresses within a fracture zone may differ significantly from those that occur in the surrounding, more competent rock. An investigation into the causes of the re-distribution of stresses in fracture zones is beyond the scope of this study and the forthcoming analysis is based on the stress relationships proposed for the Simpevarp site descriptive model /SKB, 2002a/.

For the stability analysis to be conducted in this study, an analytical solution is to be employed to calculate tunnel deformations, where a hydrostatic *in situ* stress field (σ_o) is assumed. For this study, a range in the hydrostatic *in situ* stress (σ_o) for each depth is to be used in order to take into account the effects of variations in the stress. The assumed ranges in the hydrostatic stress at different depths are displayed in Table 2-6. These values are based on Table A1-13 in Appendix 1, using the minimum and maximum stress values for each depth.

It is worth pointing out that numerical calculations are often used for detailed tunnel stability analyses. In such numerical models, suitable values of rock mass strengths and *in situ* stresses must be adapted to avoid the plasticity of the rock mass under initial conditions.

Table 2-6. Hydrostatic stresses to be used in the descriptive model.

Depth (m)	σ_o (MPa)
200	5–16
400	10–29
600	14–43

2.5 Rock mass properties

The following rock mass properties are required for the descriptive model.

- Mohr-coulomb parameters of the rock mass (e.g. cohesion and friction angle).
- Modulus of deformation of the rock mass.

Because of the large scale, rock mass properties cannot be measured directly. Empirical relations and numerical simulations may be used for the estimation, but both approaches contain significant uncertainties. Final selection of the parameters involves evaluation of the results from different approaches combined with judgement /Andersson et al. 2002/. In the case of this study, the empirical approach is employed, where empirical relations based on different rock mass classification systems are used. The commonly used systems are the Rock Mass Rating (RMR) system /Bieniawski, 1989/ and the Tunnelling Quality Index (Q) system /Barton et al. 1980/. There are also other rock mass classification systems such as the Geological Strength Index (GSI) /Hoek, 1997/, the RMS /Stille et al. 1982/, and Rock Mass Index (RMi) /Palmström, 1995/. All these methods have been discussed by /Röshoff et al. 2002/.

Data obtained from the passage of deformation zone NE-1 is to be used for the estimation of the rock mass properties. The review of the SKB's report (see Appendix 1) shows that the RMR system was used to classify the rock mass, however, it was not mentioned in the reports as to which version of RMR was used. Since the Äspö-HRL was constructed in the early 90's, it is presumed that RMR_{89} was used. But, it was not possible to obtain the raw data for the parameters (e.g. RQD), which form the basis of the system. Furthermore, no laboratory test data conducted on samples coming directly from or in close proximity to NE-1 was found. The RMR-values determined by the geological mapping are presented in Table 2-7 /Markström et al. 1996/.

In view of the uncertainty surrounding the rock mass quality of deformation zone NE-1, and taking into account the character of the study, it was decided that the rock mass properties would be evaluated by using a combination of experience from previous underground excavation projects in Sweden /e.g. Rosengren et al. 1997; Chang, 1998/, together with a simple assessment of the limited data using the GSI method proposed by /Hoek and Brown, 1997/. The analysis of this data resulted in the values quoted in Table 2-8 being proposed for the descriptive model.

For estimation of modulus of deformation E_m given in Table 2-8, the following relationship between RMR and Young's modulus proposed by /Serafim et al. 1983/ is used.

$$E_m = 10^{\frac{RMR-10}{40}} \quad \text{(GPa)} \tag{2-4}$$

Assessment of the rock mass strength using the method proposed by /Hoek and Brown, 1997/ is undertaken and results are compared with the values traditionally used in Swedish rock engineering design. The method is essentially based around the Hoek-Brown (H-B) failure criterion /Hoek et al. 2002/, which incorporates three main input parameters; Geological Strength Index (GSI), compressive strength σ_{ci} and a constant m_i for intact rock. The two constants, σ_{ci} and m_i are best determined by statistical analysis of the results of a set of triaxial tests conducted on intact rock core samples. However, when laboratory tests are not available /Hoek and Brown, 1997/ suggest that σ_{ci} can be determined from simple index testing and m_i directly from a correlation with the rock type in question. GSI can be derived directly from RMR_{89} where GSI is > 25 using the equation GSI = RMR_{89}–5, where RMR_{89} has the groundwater rating set to 15 and the Adjustment for Joint Orientation set to zero. Typical values of these parameters for different rock qualities are given in Table 2-9, according to /Hoek, 2000/.

Table 2-7. RMR-values for deformation zone NE-1 from tunnel mapping /Markström et al. 1996/.

Chainage	Description	RMR
1/285–1/298	Fair	41–60
1/298–1/301	Poor	21–40
1/301–1/303	Very poor	< 21
1/303–1/310	Poor	21–40
1/310–1/320	Fair	41–60

Table 2-8. Rock mass strengths to be used for this study.

	RMR	Cohesion (MPa)	Friction angle (°)	E_m (GPa)
Core of fracture zone	21–40	0.6–0.8	20–30	1.9–5.6
Transition zone	41–60	0.8–1.6	30–40	5.6–17.8

Table 2-9. Typical values for the parameters used for H-B failure criterion /Hoek, 2000/.

	GSI	σ_{ci} (MPa)	m_i
Very poor quality rock mass	30	20	8
Average rock mass	50	80	12
Very good quality hard rock	75	150	25

The procedure proposed by /Hoek and Brown, 1997/ was followed using the limited data obtained from the review of NE-1. The input values for the parameters used to obtain rock mass strength in terms of Hoek-Brown parameters are given in Table 2-10. GSI-values were calculated using the equation GSI = RMR-5, where RMR is used instead of RMR_{89} owing to the lack of base data for the individual RMR parameters for NE-1.

In the absence of laboratory testing conducted on samples from, or in close proximity to NE-1, it was decided that the parameters m_i and σ_{ci} would be determined using the simplified method of a comparison with the rock types in question. Intact rock strength (σ_{ci}) for hard crystalline rocks such as gneiss and granite typically lies between 100–250 MPa and the Hoek-Brown constant m_i may lie in the region 25–33. This is in agreement with the values quoted by /Hoek and Brown, 1997/ for a very good quality hard rock mass, as presented in Table 2-9. However, these values can be significantly reduced if the rock has been subjected to a significant degree of secondary alteration and is cut by veins of relatively low strength minerals such as chlorite, calcite etc. The RMR values quoted for the transition zone (40–60) in Table 2-7 suggest that the rock mass is of fair/average quality. /Hoek and Brown, 1997/ provide typical values of σ_{ci} = 80 MPa and m_i = 12 for an average rock mass (see Table 2-9). Consideration of these values, together with the fact that the intact rock strength is likely to have been reduced in the transition zone due to veining and alteration, and that the RMR values (which incorporate a parameter for intact rock strength) are relatively low, mean that the rock cannot simply be treated as an ordinary, fresh granite. In ideal circumstances the parameters would have been determined through triaxial testing of intact rock samples, however in their absence, engineering judgment has been used to allocate minimum and maximum values of σ_{ci} = 90–110 MPa and m_i = 20–25 for rock in the transition zone.

Intact rock properties for the core of NE-1 are even harder to estimate than for the transition zone. An extreme variation in geology, ranging from soft clay gouge to strong granite blocks abutting against each other fits the descriptions of NE-1 and other brittle deformation zones encountered in crystalline rocks. As a result, a significant degree of engineering judgment must be applied when applying rock mass parameters to such heterogeneous rock masses. /Hoek and Brown, 1997/ also give typical values of σ_{ci} = 20 MPa and mi = 8 for very poor rock masses (see Table 2-9). However, these values are considered to be too low for the core of fracture zone NE-1, as they are felt to be more attributable to extremely weak, more homogeneous rock types such as graphitic schist and phyllite. After some consideration, engineering judgment has been used to allocate minimum and maximum values of σ_{ci} = 70–90 MPa and m_i = 15–20 for the rock in the core zone.

Table 2-10. Estimated values for the parameters used for H-B failure criterion based on data from NE-1.

	RMR	GSI	σ_{ci} (MPa)	m_i
Core of fracture zone	21–40	16–35	70–90	15–20
Transition zone	41–60	36–55	90–110	20–25

A comparison of the failure envelope derived from the Hoek-Brown failure criterion (using the parameters presented in Table 2-10) directly against the Mohr-Coulomb criterion using the parameters given in Table 2-8 is shown in Figure 2-3 for the transition zone and in Figure 2-4 for the core zone. The figures underline the difficulty of fitting the linear M-C criterion to the non-linear H-B criterion for such a wide range of minimum principal stress σ_3, noting that the "working range" of σ_3 for a tunnel lies between nil close to the tunnel perimeter and the initial stress σ_o in the far field. Nevertheless, the figures demonstrate that these failure envelopes are in broad agreement, whilst the Mohr-Coulomb criterion being somewhat lower relative to the Hoek-Brown criterion.

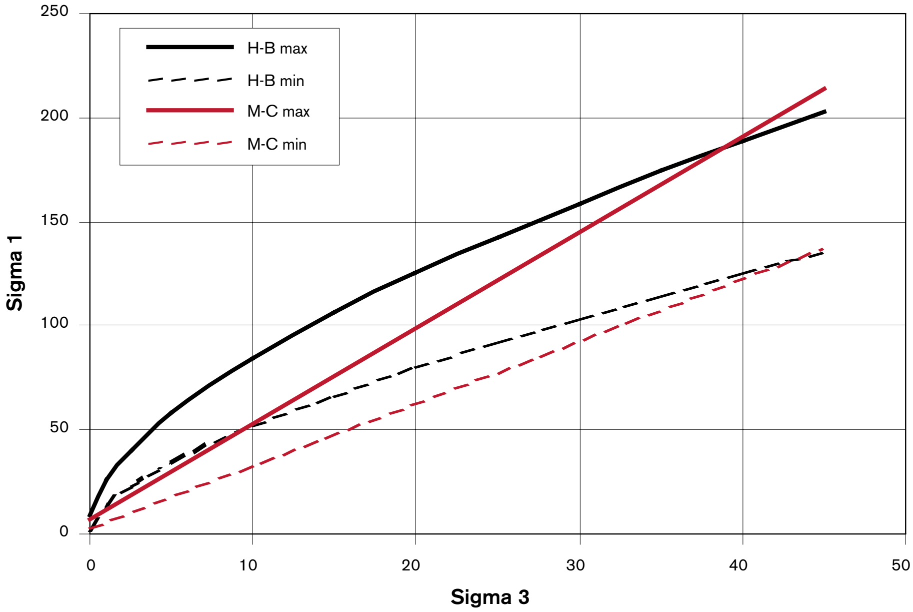


Figure 2-3. *Comparison between the H-B and M-C failure criterion for the rock mass in the transition zone.*

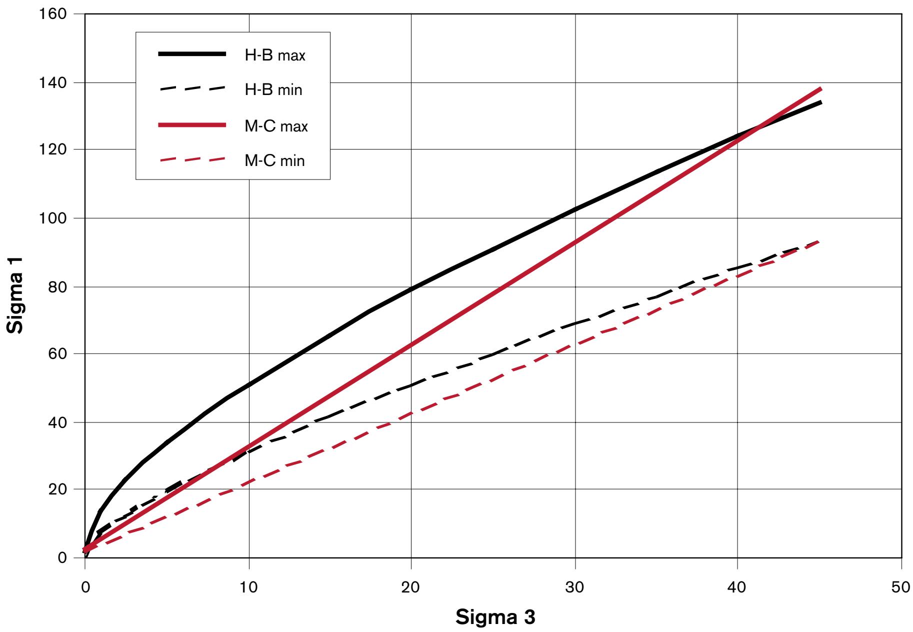


Figure 2-4. *Comparison between the H-B and M-C failure criterion for the rock mass in the core of the fracture zone.*

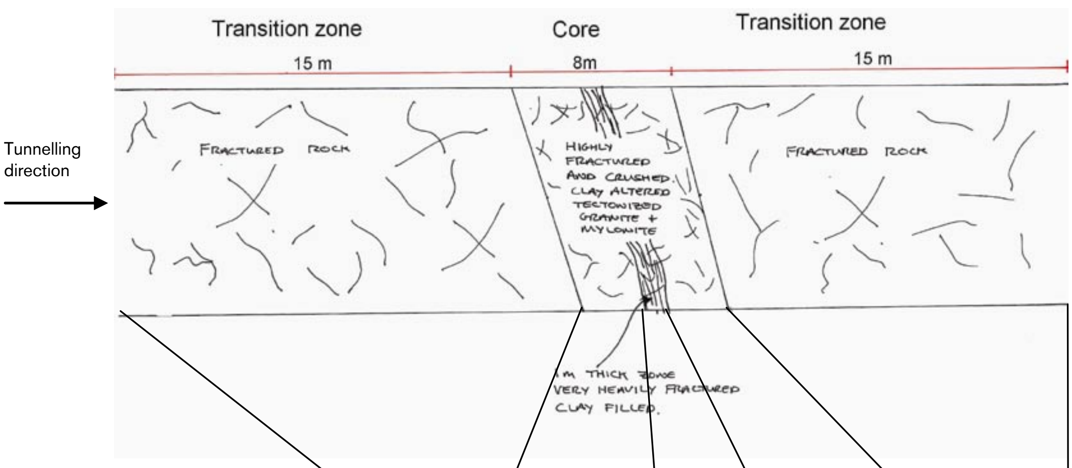
With the consideration of the experiences from underground excavation projects in Sweden and the comparison with the GSI-method as given above, it can thus be concluded that the strength values as given in Table 2-8 are considered reasonable for the purposes of this study. In view of the uncertainty as discussed above, the values of the rock mass properties as shown in Table 2-8 are specified in ranges, with the purpose to incorporate the uncertainty in the analysis.

The discussion given above demonstrates that estimation of rock mass properties is a problematic issue and is strongly dependent on the method(s) used. This raises the issues of which methods are suitable for determining rock mass properties in Swedish crystalline rock. The issue is clearly of significance for rock engineering design and it has been stressed by /Andersson et al. 2002/ and /Hudson, 2002/. It is stated by /Hudson, 2002/ that the difficult task of predicting properties in deformations zones should be recognized and, if possible, data should be collected in the vicinity of the planned location of the excavations.

2.6 Summary of the descriptive model

A summary of the descriptive model for a water-bearing fracture zone is presented in Figure 2-5. This model will be used as the basis for the analyses in this study. The 1 m thick clay zone is to be treated as a part of the core zone and not as a separate entity. Therefore no separate properties have been allocated to it. This is due to the fact that during the driving of the access tunnel for Äspö HRL, no specific problems were experienced in the clay zone aside of the those encountered in the rest of the core zone. In addition, no test results or data related specifically to the clay zone have been found.

Properties regarding the geological, mechanical and hydrogeological conditions are to be used for all the three depths. Different values of the hydrostatic *in situ* stresses σ_o as given in Table 2-6 will be used for each depth.



E-modulus, GPa	5.6–17.8	1.9–5.6	–	1.9–5.6	5.6–17.8
Cohesion, MPa	0.8–1.6	0.6–0.8	–	0.6–0.8	0.8–1.6
Friction angle, Deg	30–40	20–30	–	20–30	30–40
Hydraulic Conductivity K, m/s	Min: 2.0E–6 Mean: 1.8E–5 Max: 4.9E–5	Min: 1.2E–5 Mean: 8.8E–5 Max: 2.2E–4			Min: 2.0E–6 Mean: 1.8E–5 Max: 4.9E–5

***Figure 2-5.** The descriptive model of the waterbearing fracture zone to be used in this study (sectional view).*

3 Overview of engineering issues

3.1 General

The objective of this chapter is to identify technical issues associated with tunnelling through a water-bearing fracture zone. General descriptions of the technical issues are to be given in this chapter, while more detailed analyses will be presented in Chapters 4 and 5.

Engineering performance was documented during the driving of the access tunnel through NE-1. A summary of the experience gained from the passage is presented in section 3.2. More details can be found in Appendix 1.

A review of relevant case histories has been conducted in order to assess how other projects have tackled similar ground conditions to those encountered during the passage of NE-1. Details of these case histories are given in Appendix 2, and a short summary is given in chapter.

A system analysis is given in this chapter, as tunnelling through a water-bearing fracture zone can be described as a complex system where various components are coupled and interact with each other.

3.2 Engineering issues encountered during tunneling through NE-1

The following summary of the engineering issues encountered during tunnelling through NE-1 is exclusively based on the findings in SKB's reports. In order to improve readability, the report references have not specified here, however, a complete review, together with references can be found in Appendix 1.

The major difficulty experienced during the passage of NE-1 was related to the grouting and drilling operations due to the fractured nature of the rock mass and the relatively high water pressure. Initially grouting was carried out with restrictions on the maximum volume of grout that was to be pumped into each hole and the injection pressure in order to limit the extent of the grouted zone. In addition, the initial concept was based on relatively short grout holes that did not penetrate the entire deformation zone as well as using a grout mix that had a relatively low initial strength. This concept was planned with the aim of minimizing impact on hydrological conditions.

Due to the unsatisfactory sealing results of the grouting, changes were made to the grouting strategy. Tests were conducted to determine the optimum grouting fan, injection pressure, grout medium and initial grout strength. It was found that long grouting holes penetrating the whole fracture zone and grouts with higher initial strengths gave more effective grouting results. Grouting holes with high water-cement ratios did not have desired sealing effects. Theoretical and practical evidence also indicated that the grouting pressure needed to exceed twice the water pressure to provide a sufficient dispersion of the grout cement. A new grouting concept based on these findings was then employed and the water inflow was reduced to an accepted level. The tunnel excavation proceeded at a relatively rapid pace of about 2 m per day after implementation of the new grouting strategy.

Problems were encountered during the drilling due to water pressure acting on the drill rods and pushing them out of the hole at high speeds. This resulted in dangerous working conditions for the drill crews and caused partial flooding of the drill site. Introducing a special drilling arrangement that included a combination of drill niches, pump niches, borehole casing and a special water valve solved the problem. When the drill rod was extracted, the valve was closed in order to limit the inflow of water from the borehole, which meant that the drill site remained relatively safe and dry. The system also improved the possibility to measure flow rates from the borehole.

No indications of tunnel instability have been noted in the reports with regard to the passage of deformation zone NE-1. However, concerns relating to high water pressure destabilizing the tunnel face, particularly during the drilling and grouting process have been noted. In order to guard against this, the distance between the tunnel face and deformation zone NE-1 was kept to about 20–30 m to ensure face stability during the drilling and grouting operations. The boreholes drilled from the face were cased to prevent water from being introduced to the fractures in the vicinity of the face. Rock support consisted of fibre-reinforced shotcrete, rock bolts and steel mesh installed in the tunnel walls and roof, no details have been found which suggest the use of face support. No details of deformation measurements were found in the SKB's report.

3.3 Summary of case histories

The study of case histories is included in this study to provide an insight into tunnels that have encountered similar problems/ground conditions as those concerned in this study. The details of the case histories are given in Appendix 2 and the following presents a short summary.

It is evident that the method selected for tackling difficult rock conditions is case specific. What is clear is that characterization of the zone and preparation of a "plan of attack" prior to exposing the zone in the tunnel face is critical. Lack of sufficient site investigation is sited as a cause for the problems encountered at the Linköping petroleum storage project and for the Vexin tunnel project. The use of advance probe drilling to locate and characterize weakness zones is a particularly vital part of the investigation process and was used at Oslofjord, Bjorøy and at the Orange Fish tunnel. In addition, where water-bearing fracture sets are suspected, it is important to ascertain their orientation during the probing in order to avoid driving the tunnel parallel to a major water-bearing fracture system, as was the case in the Vexin tunnel.

Practical issues, such as ensuring that sufficient pump capacity for the largest sudden inflow is available, are important to maintain a safe working environment, and to ensure that the tunnelling process is as effective as possible. The working sites at both the Vexin tunnel and the Orange Fish tunnel were flooded due to insufficient pump capacity.

Modification of the tunnel route to minimise the length of the tunnel through the fault zone (e.g. Jonkershoek tunnel system), driving of bypass tunnels (e.g. Oslofjord) and adits/niches (e.g. Linköping petroleum storage), to work multiple faces can speed up and assist the passage of weakness zones.

There are a multitude of combinations to overcome overstressed ground and sections of poor rock. Reduction of the size of the working face (multiple headings/top heading and bench), have been employed on many tunnel projects, such as Samanalawewa, Tuzla and the Linköping petroleum storage in order to improve control of tunnel stability. Use of a forepole umbrella to provide support to the ground ahead of the tunnel face was employed at Tuzla, Bjorøy, Nathpa-Jhakri and at the Jonkershoek tunnel system, however, /Hoek, 2001/ warns about the problems associated with using forepoles which are too long and overstress the tunnel support at the face (commonly steel sets). Provision for the increase in load at the base of steel sets can be made to avoid foundation failure. Support of the tunnel face in poor ground in the form of shotcrete and dowels (Bjorøy) can be used to control face stability. Alterations of the tunnel's form (Samanalawewa) and reinforcement of the invert (Bjorøy) are methods that enable the tunnel to accommodate high stress environments. Self-drilling rockbolts can be used in poor rock where drilling is difficult. Steel sets are commonly used (e.g. Tuzla) in conditions where rockbolts cannot gain sufficient anchorage. Tensioned, grouted cables offer an alternative to rockbolts where heavier support is required or where the distance to a firm anchorage is larger than normal. Grouted cables were successfully used in the Mucha Highway tunnel and the Inntal tunnel. In situations where extremely large deformations are anticipated, the support may need to be designed to accommodate a degree of deformation before it becomes active. Such examples include the Yacambú-Quibor tunnel (steel sets) and the Inntal railway tunnel (shotcrete).

Drainage holes have been used in several projects to reduce the local hydraulic gradient and improve drilling conditions. Examples in which drainage methods were employed include Samanalawewa, Vexin, Bjorøy and Tuzla.

Fault zones may contain swelling minerals that can exert large pressures on tunnel linings if no provision for them is made. If shotcrete is applied too rapidly, swelling minerals are unable to expand and hence apply pressure to the lining, which can lead to failure at a later date (e.g. Rafnes).

Pre-grouting are commonly used for tunnelling in weakness zones. Projects in which pre-grouting was used include Tuzla, Vexin and Linköping. A preliminary phase of pre-grouting, prior to ground freezing was also used at Oslofjord. The Bjorøy tunnel project used a 2 stage grouting process in poor rock and 4 different grout types in soil sections. Difficulties associated with the drilling process were tackled by using sectional grouting with steel pipes.

Two projects in which grouting failed due to high water pressures and bad ground conditions were a section of the Jonkershoek tunnel system and the Oslofjord crossing. As a result, ground freezing was employed in both projects to facilitate the passage of these zones.

3.4 System description

As describes in the previous sections, tunnelling through a water-bearing fracture zone under both high water pressure and high rock stresses is a difficult task. In order to provide an overview of the entire technical system for tunnelling under such conditions, a system analysis has been conducted, aiming at to identify the relationships between the key components involved in the system. One method for describing such a complex system is an object relation diagram using Unified Modelling Language (UML) /Eriksson et al. 2000/, where the relations between the key components of the technical issues are visualised.

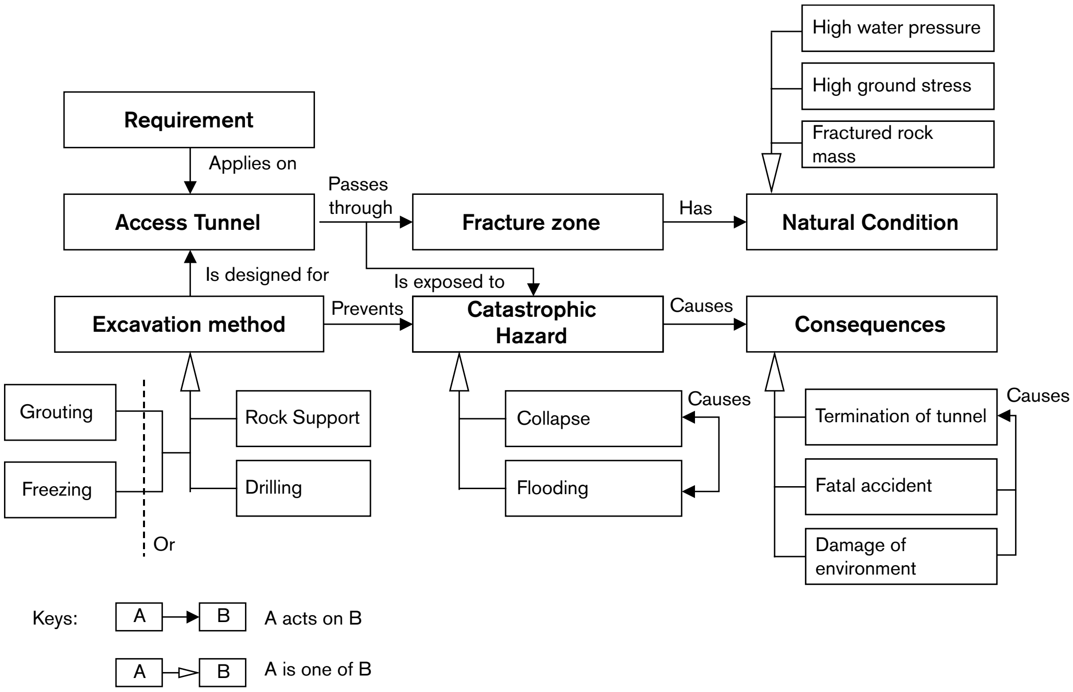


Figure 3-1. *System description using the UML (Unified Modelling Language) for passage of water-bearing fracture zone.*

The system description constructed for this project is shown in Figure 3-1. An interpretation of Figure 3-1 is given below:

The access tunnel will pass through a fracture zone that causes problematic tunnelling conditions caused by high water pressure, high ground stress and fractured rock. The passage through such a fracture zone is exposed to potential hazards consisting of tunnel collapse and flooding. A tunnel collapse might cause flooding into the tunnel and vice versa. An event due to the occurrence of a hazard could lead to one or more consequences such as termination of the tunnel project, a fatal accident or damage of the environment. To enable a safe passage through the zone an excavation method must be designed to prevent such hazardous events.

As a result of the system analysis, the technical issues associated with tunnelling through a water-bearing facture zone can be roughly divided into the following categories:

A. Issues related to control of water inflows by grouting or ground freezing;

B. Issues related to tunnel stability;

C. Other issues related to achieving a safe and efficient tunnel construction.

To identify the important technical issues, a so-called "matrix method" has been employed. In the matrix the impacts of the natural tunnelling conditions (e.g. high water pressure, high ground stresses and fractured rock mass) are evaluated for each category of the technical issues. The identified technical issues are presented in Table 3-1 and will be described in general terms in section 3.5.

Table 3-1. Technical issues associated with passage of the water-bearing fracture zones.

Category of technical issues	Tunnelling conditions		
	1 **Fractured rock mass**	**2** **High water pressure**	**3** **High *in situ* stress**
A Water inflow issues	• High permeability of rock mass	• Hazards by high water inflow • Difficulties in grouting to achieve required sealing effects	• Rock movements causing leakage in grouted/frozen zone
B Stability issues	• Low rock mass strength • Falling blocks • Swelling clay in fractures	• Flowing ground • Stability of grouted/ frozen zones • Water pressure on tunnel lining • Water pressure on rock blocks, including blocks in the tunnel floor • Face stability • Short stand-up time	• Failure due to over-stressed rock mass • Face stability • Short stand-up time • Local brittle rock failure and ravelling along discontinuities
C Constructions issues	• Collapse of the borehole wall	• High pressure drilling; • High water flow from boreholes • High pressure grouting	• Large deformation requires over-excavation • Choice of rock support procedures • Excavation procedures

3.5 General description of technical issues

3.5.1 Water inflow related issues

Large water inflows may result in hazardous working conditions during tunnel excavation and high operational costs during tunnel operations. In the worst-case scenario water inflows combined with water pressure acting on the rock mass/tunnel support, may result in tunnel collapse, e.g. flowing ground during tunnel excavations. Control of the water inflows is therefore of essential importance for the safe passage of a water-bearing fracture zone.

The commonly used methods for water inflow control are grouting and ground freezing. The choice between grouting and ground freezing is not always easy and the decision should be made based on careful evaluation of the hydrogeological conditions, costs and time schedules.

For grouting in fractured rock under high water pressure, the important issues are requirement specifications and grouting design with considerations of e.g. the rock mass structures, types of grout and grouting pressures. Stability of the grouted zone must be considered under high water pressure.

For ground freezing, the key issues are the thermal design and structural design. Though the primary purpose of freezing is to control the water inflows, the stability of the frozen zone must be ensured. More detailed analysis and discussion about water inflow related issues will be given in Chapter 4.

3.5.2 Stability issues

The major stability issues for the passage of the water-bearing zone are:

- Instability of the tunnel face;
- Instability of tunnel roof and walls.

Generally speaking, instability of a tunnel is often caused by 1) overstressing of the rock mass; and/or 2) structurally controlled failure. Instability of the tunnel face may occur due to the effects of water pressure acting on discontinuity surfaces in front of the tunnel face. /Hoek et al. 1995/ presented a guideline for identifying rock mechanical problems associated with different rock mass characteristics subjected to different stress conditions (Figure 3-2).

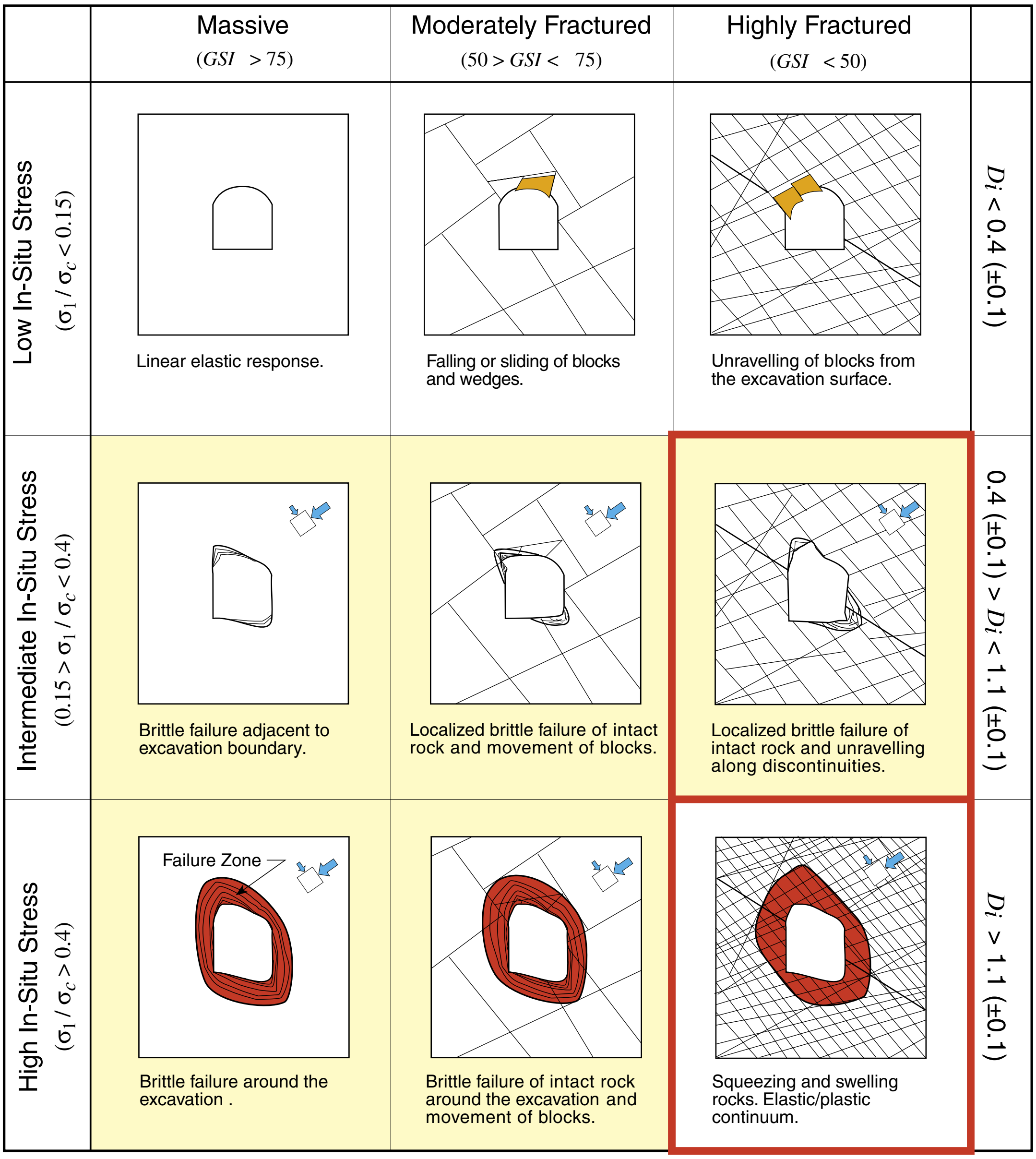


Figure 3-2. Behaviour of different rock masses and stress conditions concerning stability issues /Hoek et al. 1995/ and modified by /Martin et al. 2001/.

Figure 3-2 is used to outline the potential rock mass behaviours for the ground conditions in this study. The rock mass parameters presented in the descriptive model indicate that the majority of the deformation zone has a GSI value < 50 and falls in the category "highly fractured rock". Stress-strength ratios σ_1/σ_c are estimated by assuming the parameter "strength of intact rock" (σ_c) to be 100 MPa and the values of the maximum principal stresses (σ_1) to be 16 MPa at 200 m depth, 29 MPa at 400 m and 42 MPa at 600 m. It should be noted that these parameters are in accordance with the descriptive model given in Chapter 2. The ratio of σ_1/σ_c obtained for the three depths indicates that the "intermediate in-situ stress" condition will apply at depths of 200 m and 400 m while the "high in-situ stress" condition can be applied at 600 m.

Using the behaviours outlined in Figure 3-2, it is proposed that the rock mass behaviours to be considered for this study are:

1. Overstress of fractured rock mass (squeezing).
2. Localized brittle failure of intact rock and ravelling along discontinuities.

It is worth noting that "squeezing" commonly implies time-dependent behaviour of a rock mass. For the purpose of this study the term "overstressed fractured rock mass" is to be used instead of "squeezing", implying large tunnel deformations due to the highly fractured rock mass and high stress levels.

Based on the past experiences and the case histories, it is considered that localized brittle failure of intact rock is not a critical situation to be included in this study.

The structurally controlled failures, such as sliding or ravelling along discontinuities caused by gravity or rock stresses, can cause downfalls of rock blocks. In order to provide a preliminary indication of the degree of difficulty associated with the structurally controlled problems, block analysis is to be undertaken to estimate volumes of potentially unstable blocks. In the design of the rock supports in the later stages of the SKB's design process, however, all forces acting on the blocks (e.g. water pressures) must be taken into consideration.

Swelling ground associated with swelling clay might lead to post-construction failures in tunnels. The case histories presented in Appendix 2 indicate that swelling minerals in deformation zones can exert large pressures on tunnel linings if no provision for them is made. This may result in failure of the lining at a later date. However, further analysis of the problems associated with swelling clays will not be included in this report due to the absence of data regarding the clay in NE-1 and the fact that there is no evidence available that suggests that swelling related problems have occurred in the access tunnel at the Äspö HRL (see Appendix 1).

In summary, more detailed analysis of the following stability issues will be undertaken in this study:

- Stability associated with large deformations due to overstressed fractured rock mass;
- Stability of tunnel face;
- Structurally controlled failures.

In addition, stability of grouted/frozen zones will also be included. The analyses of these stability issues are given in Chapter 5.

3.5.3 Construction issues

As indicated in Table 3-1, construction issues to be considered for tunnelling under the given conditions are drilling operations, probing, grouting arrangement, rock support and excavation procedures.

During conventional drill and blast tunnelling, drilling is required to perform numerous activities, including probing, grouting, installation of bolts etc. As has been highlighted in section 3.2, drilling under high water pressure in fractured rocks is a difficult task. In certain circumstances special measures must be taken prior to entering the zone in order to ensure the safety and efficiency of the drilling operations.

During tunnel construction, probing of the fracture zone in advance ahead the tunnel face is critical. Where water-bearing fracture sets are suspected, it is important to ascertain their orientation during the probing in order to avoid driving the tunnel parallel to a major water-bearing fracture system.

The installation of traditional packers in grouting holes may not be possible due to the water pressure acting on the packers. During the passage of NE-1, sliding of the packers along the grouting hole due to the high grouting pressure occurred and proved to be problematic. Special arrangements for grouting under such conditions by using for example a T-valve have been reported in some of the projects presented in Appendix 2.

Tunnels in fractured rock conditions often have limited stand-up times, and therefore require rapid installation of tunnel support. When shotcrete is used, the loading process on the shotcrete lining associated with tunnelling procedures must be considered with regard to the curing process of the shotcrete. Over-loading of the shotcrete lining prior to it reaching its maximum strength will result in a deterioration of the quality of the lining.

The construction related issues will be discussed in Chapter 6.

4 Grouting and ground freezing for water inflow control

4.1 General

The aim of this chapter is to discuss the technical issues associated with water inflow control by the use of grouting or ground freezing. In the following sections, technical issues and possible solutions related to grouting and ground freezing are discussed. The section is essentially intended as an overview of these possible solutions. Detailed design of a specific grouting or ground freezing solutions require further investigation and should be included in the coming stages of SKB's design process.

4.2 Grouting in fractured rocks

Grouting of fractured and highly conductive rock masses to seal fractures and hence reduce water inflow into tunnels is a complex process. Despite intensive research work in Scandinavia over the last few decades, the process of injecting grout into a fractured, heterogeneous rock mass is still not comprehensively understood. Laboratory and field experiments have been conducted and theories have been developed to understand the mechanism of flow of grouts in rock fractures e.g. /Hässler, 1991; Jansson, 1998; Ericsson, 2002/.

In this section, a general description is given on the methodology for grouting design, which is commonly practised in Sweden. Simplified analyses are then presented for estimation of water inflows in relation to grouting effects in terms of improved hydraulic conductivity of the grouted zone. An indication of the suitability of grouting as a mean for water inflow control can therefore be gained by analysing the required hydraulic conductivities for a given allowable water inflow. Finally, suggestions for possible grouting solutions are presented. The analyses and discussions given in this section are based on the assumption that cement-based grouts are to be used.

4.2.1 Methodology for grouting design

One of the most commonly used methodologies for grouting design practised in Sweden is shown in Figure 4-1. The grouting design is based on a requirement specification covering:

- Environmental requirements regarding the influences on the ground water conditions etc;
- Functional requirements for tunnel operations regarding allowable water inflows;
- Durability of the grout material, for example the chemical composition of the grout in relation to the ground water chemistry;
- Functional requirements for tunnel construction regarding working environment and stability of the tunnel;
- Production requirements regarding the time for grouting operations, grout setting time and number of grouting rounds.

Figure 4-1. *Outline of the methodology for grouting design /modified after Hässler et al. 2004/.*

Swedish tunnel projects are already subjected to specific environmental and functional requirements, which may vary depending on the location of the tunnel. These specific requirements must be determined prior to the tunnel construction. These requirements will have a significant impact on the design of the grouting program.

The specification of the allowable water inflows should ideally take into account the site-specific hydrogeological conditions. Specified allowable water inflow must take into consideration what can be achieved practically and economically.

In general, the goal of grouting design is to describe the methodology to achieve the required permeability of the rock mass surrounding the tunnel. Site testing and continuous modifications are often required in order to be able to cope with the actual hydrogeological conditions. The resulting design is usually a specification containing the following components:

- Fan geometry and overlap. It may be necessary to have several grouting fans applied in sequence from the same grouting location;
- Minimum drilling diameter and requirements for drilling precision;
- Requirements for colloidal mixer and pump capacity;
- Requirements for grout types and properties including cement type, cement particle sizes and distributions, additives such as super plasticizers and accelerators, grout properties such as separation stability, filtering stability, cohesion, viscosity, hardening characteristics and final strength. Several different grouts are usually specified for use in different situations;

- Hydraulic tests before grouting, such as measurements of inflow or lugeon (packer) tests;
- Grouting procedure including grouting sequence (the order in which the holes should be grouted), use of grout types under different conditions, maximum and minimum pressure at different stages of the works,
- The minimum/maximum grouting volume and minimum/maximum grouting time for each hole in each fan;
- Criteria for termination of grouting;
- Definitions on when drilling can be started after a fan is finalized (which usually depends on the grout's hardening characteristics);
- Requirements on mixing time and testing of grout properties;
- Hydraulic tests to be performed to control that sufficient sealing is achieved.

Generally speaking, there are two grouting techniques, namely high grouting pressure using stable grouts (lower W/C ratios) and low grouting pressure using thin grouts (high W/C ratio). Based on past experience in Sweden and Norway, it is the authors' opinion that the technique of high grouting pressure with stable grouts (lower W/C ratios) is more suitable for rock masses of high hydraulic conductivity under high water pressure. This technique has the advantage that the stable grouts have rapid hardening rates as well as high shear strengths, which provides good resistance to "wash-out" due to the effects of water pressure. Grouting pressure can also have a significant effect on the sealing of the rock mass. It is generally accepted that an increase in the grouting pressure enhances the grout takes and thus results in a better sealing effect /Karlsrud, 2003/.

The major limitation of increasing the grouting pressure is that it may introduce so-called hydro-jacking or hydro-fracturing during grouting /Lombardi, 2003/. Hydro-jacking refers to the opening of the existing fractures while hydro-fracturing refers to the opening of new fractures during the grouting processes. Clearly the re-opening or introduction of new fractures is detrimental to the grouting process. A concept using "grouting intensity" can be used to assess the likelihood of hydro-jacking and hydro-fracturing occurring during grouting. A so-called "Grouting Intensity Number" or GIN-value, defined as a product of grouting pressure and grout volumes, is introduced for controlling the grouting process. Three limits are taken into account according to the GIN-principle /Lombardi, 2003/:

1. Maximum pressure;
2. Maximum grout take or grout volume;
3. Maximum grouting intensity (GIN-value).

The maximum pressure must be evaluated on a continuous basis and consideration should be given to local tunnel conditions. Under poor rock conditions, maximum grouting pressure must be limited to avoid damage to the rock mass, however, the allowed maximum grouting pressure must be greater than the ground water pressure. /Lombardi, 2003/ recommends a ratio of 2 to 3 in respect of the water pressure to injection pressure, while /Garshol, 2003/ states that the allowed maximum grouting pressure should be at least 50 bar (5 MPa) above the static ground water head unless other limiting factors exist. However, /Lombardi, 2003/ and /Garshol, 2003/ do not specify if friction losses in the grouting equipment are considered for determination of the maximum pressure.

4.2.2 Analysis of grouting effects

The assessment of the effect of grouting is to be conducted using equation 4-1 /Albert and Gustafson, 1983/:

$$q = \frac{2 \cdot \pi \cdot K_g \cdot H}{\ln\left(\frac{R+I}{R}\right) + \frac{K_g}{K} \cdot \ln\left(\frac{2 \cdot H}{R+I}\right) + \xi} \tag{4-1}$$

where

q = Water inflow (m^3/s/m);

H = Water head (m);

R = Tunnel radius (m) and is set to 4 m;

I = Extension of grouted zone (m);

K_g = Hydraulic conductivity of grouted zone (m/s);

K = Initial hydraulic conductivity before grouting (m/s);

ξ = Skin factor and is assumed to be zero in this study.

This equation shows how water inflow depends on the hydraulic conductivity and the extent of the grouted zone. This equation is derived by assuming a circular tunnel, which is subjected to a hydrostatic water condition in a continuous and homogenous porous medium. Though more sophisticated methods /e.g. Gustafson et al. 2004/ are available for analysing grouting effects in jointed rock masses, the simplified procedure of /Albert and Gustafson, 1983/ is felt to be sufficient for the purposes of this study.

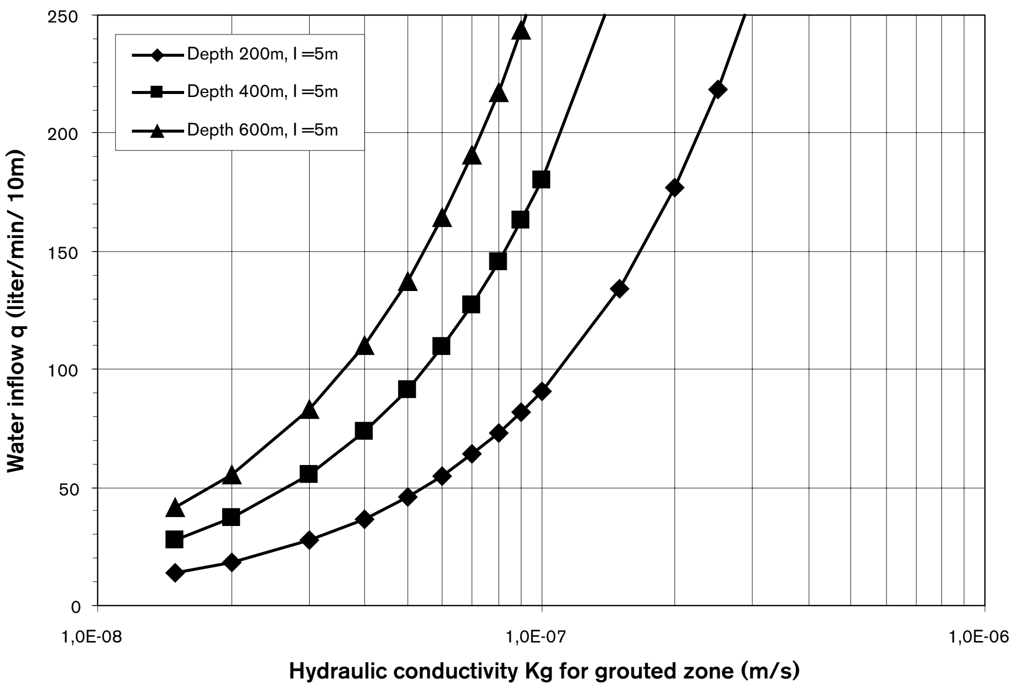
The effect of grouting in terms of improved hydraulic conductivity K_g and the corresponding reduction in water inflow is to be evaluated for the three different depths. The radius of the grouted zone is assumed to be 5 m respective 10 m. A grouted zone radius larger than 10 m is not considered to be practical from a construction point of view. The calculated relationships between the hydraulic conductivity K_g of the grouted zone and the corresponding water inflow are given in Figure 4-2 for the transitions zone. The mean value of the hydraulic conductivity as given in the descriptive model in Chapter 2 is used as the initial hydraulic conductivity K.

The required hydraulic conductivity K_g for the grouted zone can be estimated using Figure 4-2 by specifying a maximum acceptable water inflow.

During the construction of the Äspö HRL, water inflows were regularly measured at different sections along the tunnel. It was reported that the measured water inflow after the passage of NE-1 was approximately 50 l/min/10 m /Stille et al. 1993b/. Assuming that the extension of the grouted zone for NE-1 is between 5–10 m, the curves for tunnel depth = 200 m in Figure 4-2 suggest that the hydraulic conductivity of the grouted zone K_g achieved in NE-1 is likely to be in a range of 5 to 8×10^{-8} m/s.

Table 4-1 shows hydraulic conductivities K_g needed to achieve the same amount of water inflow (50 l/min/10 m) as that observed after the passage of NE-1, for the various depths and extensions of the grouted zone. The results show that the values of the required hydraulic conductivities range from 2 to 8×10^{-8} m/s. It is interesting to note that the differences in the required hydraulic conductivities K_g between the cases for I = 5 m and I = 10 m for achieving an allowed water inflow of 50 l/min/10 m are relatively minor.

In practice, there are always variations in the hydraulic conductivities achieved in grouted zones. In order to examine how sensitive the degree of water inflow is to variations in hydraulic conductivity, values of water inflow have been calculated with the aid of Figure 4-2 assuming the value of K_g varies between 2 to 5×10^{-8} m/s. The results, which are presented in Table 4-2 indicate that water inflow is sensitive to variations in the value of K_g. Relatively minor increases in K_g result in a significant increase in the volume of water inflow. This effect, unsurprisingly, increases with depth.



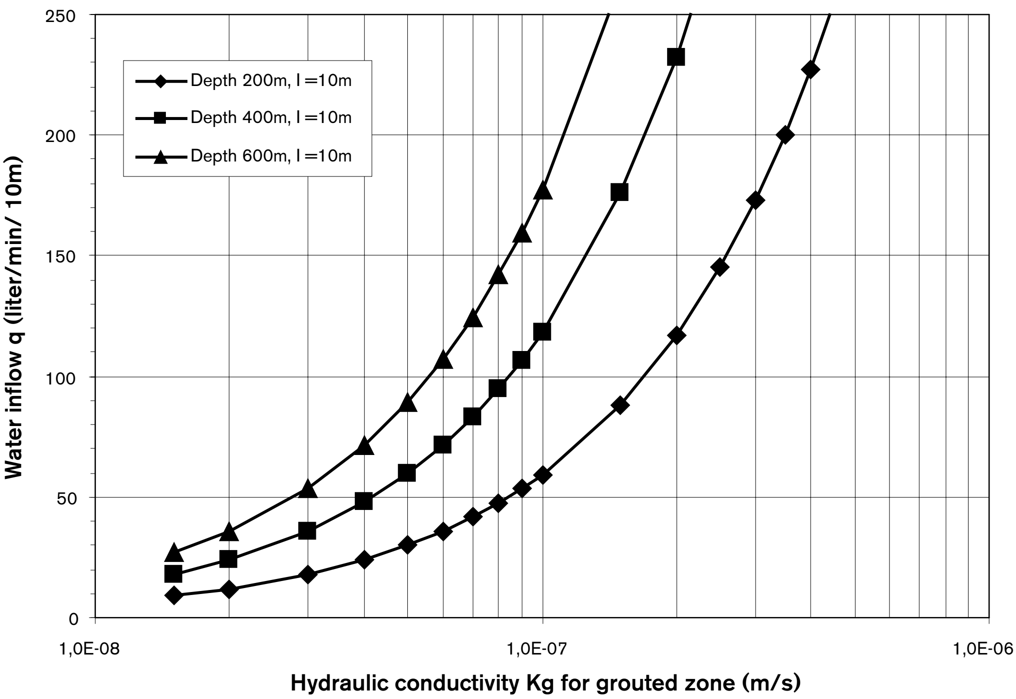


Figure 4-2. *Calculated relationship between hydraulic conductivity of the grouted zone K_g and water inflow for different depths in the transition zone (initial hydraulic conductivity K = 1.8E–5 m/s).*

Table 4-1. Estimation of required values of K_g for the allowed water inflow of 50 l/min/10 m.

Depth	For extension of grouted zone l = 5 m	For extension of grouted zone l = 10 m
200 m	5E–8 m/s	8E–8 m/s
400 m	3E–8 m/s	4E–8 m/s
600 m	2E–8 m/s	3E–8 m/s

Table 4-2. Estimated variation of water inflow (L/min/10 m) due to a variation in K_g between 2 to 5×10^{-8} m/s.

Depth	For extension of grouted zone l = 5 m	For extension of grouted zone l = 10 m
200 m	20–50	10–30
400 m	35–90	25–60
600 m	55–140	35–90

The above analysis suggests that the quality of grouting in terms of achieved hydraulic conductivity plays a more important role than the extension of the grouted zone. Though the increase of the grouted zone from 5 m to 10 m does reduce the quantity of water flowing into the tunnel, it is apparent that increasing the extension of a grouted zone, say larger than 10 m, is not an effective way to reduce water inflows.

Similar calculations have also been done for the core of the fracture zone. It is interesting to note that the results for the core are only slightly different from those calculated for the transition zone, despite the fact that the un-grouted core is more fractured and conductive. This indicates that the final hydraulic conductivity of the grouted zone has the most influence on the degree of water inflow, whereas the "initial" hydraulic conductivity, K, has little effect.

For the cases concerned in this study, the hydraulic conductivity K_g in the grouted zone must reach the order of 10^{-8} m/s for both the transition and the core of the fracture zone, in order to reduce the water inflow to 50 l/min/10 m. Such a reduction in hydraulic conductivity can be achieved using current grouting technology, however, it would require a great deal of effort to achieve this degree of sealing in the core of the fracture zone.

4.2.3 Possible grouting solutions

As mentioned in the previous section, the measured water inflows from the passage of NE-1 at about 200 m depth show that it is achievable to reduce the water inflow to a level of 50 l/min/10 m using present grouting technology. Over the past decade considerable developments in grouting technology have been achieved, particularly with regard to the introduction of high quality cement-based grouts. Despite these developments, past experiences (including the passage of NE-1) indicate that grouting in fractured rock under high water pressure is still a difficult task. For the cases concerned in this study, it will be necessary to adapt the grouting method in order to achieve the desired grouting effects for the given conditions.

An outline of one possible grouting methodology is shown in Figure 4-3. Pre-grouting is performed as close as possible to the fracture zone; even the rock mass in front of the face should be grouted to limit inflow from the face. Thereafter niches are excavated at a safe distance from the zone. Hydrogeological tests are carried out in probing holes drilled from the niches to characterize the fracture zone. Long grouting holes, which penetrate the entire fracture zone, are then drilled from the niches. Grouting is executed over the whole length of the hole. Experience from the passage of deformation zone NE-1 found that grouting in holes that penetrate the entire water-bearing zone, is a particularly effective method of sealing the rock mass (see Appendix 1).

When grouting highly fractured rock under high water pressure, it may be necessary to perform grouting in more than one round at the same location. The first round of grouting should be performed with thick grout using a high grouting pressure and is aimed at sealing open fractures. The following grouting rounds should then use thinner grouts and lower grouting pressures than the previous round to seal progressively tighter fractures. It is important to note that the grouting fans and grouting pressures for the following rounds must be designed in such a way that the successive grouting rounds avoid damaging the previous round. This issue can be investigated by using the GIN-principle. In order to prevent bleeding, lowering the viscosity of the grout should be achieved by the use of additives (such as super plasticizers) in the grout mix, as opposed to increasing the w/c-ratio.

The grouting approach outlined above can also be combined with the concept of using a “blocker” zone /Roald et al. 2003/. This requires that an initial outer “blocker” zone is first established in order to control the spreading of the grout, after which normal grouting is conducted in the inside the blocker zone (see Figure 4-4). Ideally the grout should be placed in relatively close proximity to the tunnel to form a zone with low hydraulic conductivity. The outer “blocker” zone can be created in the first round of grouting using, for example, setting-controlled cement with variable viscosities. After establishing the outer “blocker” zone, grouting using “normal” grouts can be conducted inside the “blocker” zone using a lower grouting pressure. As the “blocker” zone has a reduced hydraulic conductivity compared with the surrounding rock, the grout spread is limited, which results in a more effective grouting process.

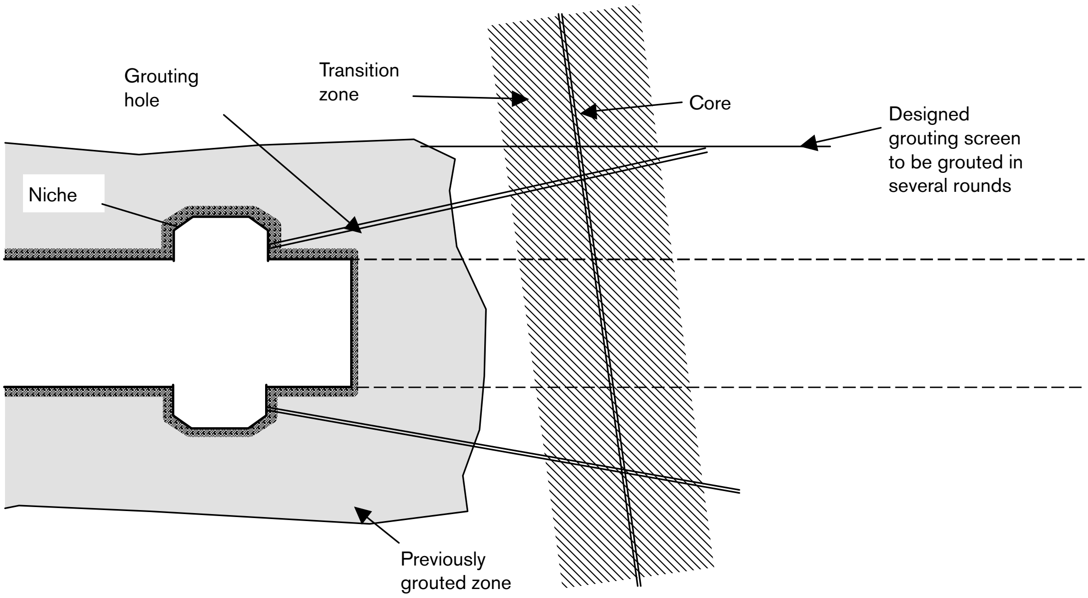


Figure 4-3. *Possible grouting strategy.*

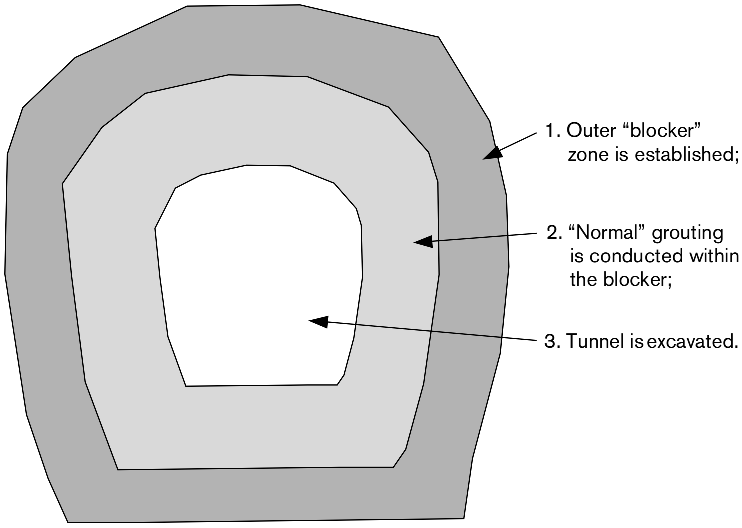


***Figure 4-4.** Illustration showing the concept of an outer "blocker" zone /modified after Roald et al. 2003/.*

The above section presents possible approaches for grouting of a fracture zone with the characteristics of NE-1. It should, however, be pointed out that the mechanism of grouting of fractured rock at depth is not yet fully understood, and a design of the grouting concept will not be completed until it has been tested at site scale.

4.3 Ground freezing

Ground freezing is a possible alternative to grouting when considering water inflow control for tunnelling. The concept is based around the fact that ground freezing can be used to temporarily exclude water from a tunnel until construction of the final lining provides a full, watertight seal. When freezing the water around a tunnel, stabilization of the ground and structural support may be achieved at the same time depending on the ground conditions. However, for the purpose of this study, the primary aim of ground freezing is to form a temporary watertight zone around the tunnel prior to the installation of the permanent lining.

4.3.1 Ground freezing in tunnels

The process of ground freezing is transient and, as such, does not affect the quality of the groundwater and has a low environmental impact. The safety record for this process is also excellent. The primary costs may appear to be high, however, when the risk and final costs are included, the method frequently proves to be the most cost-effective /Harris, 1995/.

The scale of tunnelling problems does not limit the use of ground freezing. It can readily cope with small situations and has been used for excavations up to 45 m diameter and to depths of over 900 m. The method can accommodate the full range of rock types. Generally speaking, the only limitations of the method are (1) that the rock mass must have an adequate moisture content and (2) that water-flows through or beside the intended ice-body must be nominal /Harris, 1995/.

During freezing, a frozen zone or so-called ice-wall is created around the freeze-tubes. The configuration of the freeze-tubes will depend on the size and the shape of the intended excavation. The stages in growth of the frozen zone around a tunnel are illustrated in Figure 4-5 /Harris, 1995/. The first stage is the development of individual ice columns around each freeze-tube, which subsequently merge with the neighbouring column to form a continuous hollow cylinder of nominal effective thickness. During the next stage the cusps of adjacent columns tend to smooth as the minimum design thickness is reached. A further short period may be needed for the inward ice-wall growth to reach the excavation line. The foregoing stages are called the “Active Freeze Period”. Continuous refrigeration during tunnel excavation and lining is called the “Passive Freeze Period”, as only a limited quantity of heat is removed. Security is achieved when the lining is in place. Afterwards natural or accelerated thawing can begin.

The freezing operation can be carried out in four steps, each of which is subject to detailed monitoring:

- Installation of the refrigeration plant and coolant distribution system;
- Active refrigeration to create the frozen zone;
- Refrigeration to maintain and possibly increase the thickness of the frozen zone while the excavation is made and the permanent lining is installed;
- Allowing or controlling the thaw and the effects thereof.

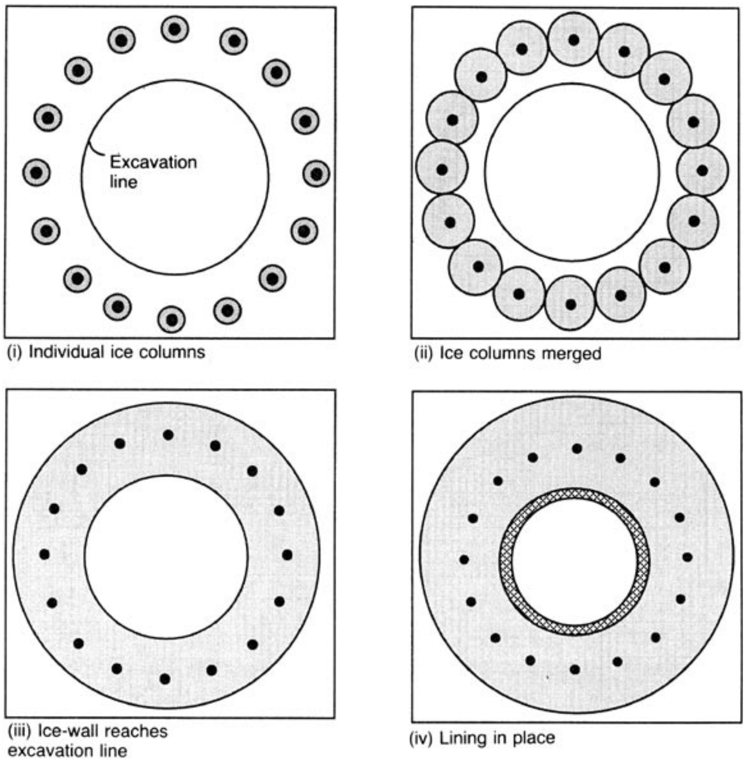


Figure 4-5. Stages in growth of frozen zone /Harris, 1995/.

4.3.2 Design methodology for ground freezing

This section will give an outline for engineering design of ground freezing, based on the work of /Harris, 1995/. In general, the design of ground freezing includes the following:

- Ground investigation;
- Thermal design to ensure sufficient freezing capacity;
- Structural design to ensure stability during freezing operations;
- Monitoring program.

During the ground investigation, the following key information should be provided and analysed:

- Mechanical and thermal properties of the rock mass;
- Ground water flow conditions;
- Salt content in the ground water;
- Ground temperatures.

The thermal and structural elements of the design are fundamental and inter-related. The task of the structural design is to ensure the tunnel stability during the tunnel excavation through the frozen fracture zone. Though the primary purpose of the ground freezing is to control the water inflow, the frozen zone around the tunnel must have sufficient bearing capacity to sustain the water pressure acting on the outer boundary of the frozen zone. When the stability issues have been analysed and the required dimensions of the frozen zone and freezing temperature determined, a thermal design can be completed. This includes determination of the optimum refrigeration loads, freeze-tube deposition, plant capacity and freezing times. The entire process including design, execution and monitoring for ground freezing is outlined in Figure 4-6.

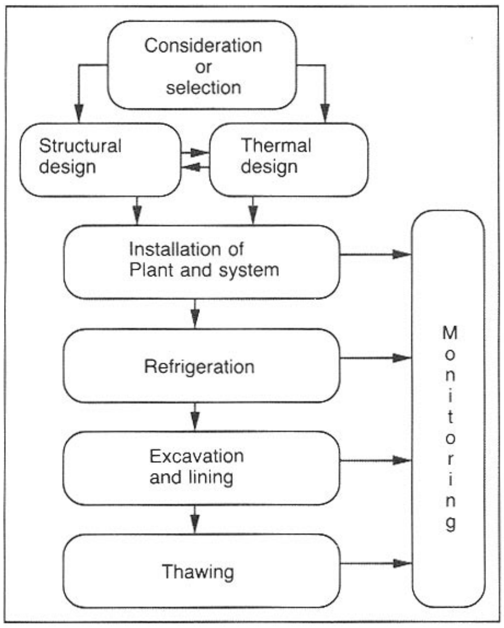


Figure 4-6. *Outline of the process for ground freezing, including design, execution and monitoring /Jones, 1982/.*

4.3.3 Thermal design of ground freezing

Groundwater flow is one of the greatest hazards to effective freezing. It is particularly important that the flow conditions are recognized and evaluated at the investigation stage. For instance, the groundwater flow due to previous tunnel excavations and grouting activities must be evaluated.

The presence of saline groundwater may also have a major impact on the freezing process. This is due to the fact that the freezing point may be depressed below zero if there are salts present in the groundwater. Trial measurements of the freezing point should be conducted if saline groundwater is anticipated. The concentration of salt (NaCl) in seawater is about 28 g/l, and the freezing point is about –2.5°C. The presence of salt also reduces the strength of the frozen material in addition to the negative effects they have on the freezing point of groundwater.

The refrigeration load, i.e. the length of the Active Freeze Period, is affected by the range of temperatures to be dealt with. For instance, the ground temperature will determine the quantity of sensible heat to be removed before reaching the freezing point, while the ambient temperature and the humidity in the tunnel will influence the performance of the freezing plant /Harris, 1995/.

The cooling process includes three distinct and successive stages: from the ambient temperature to the freezing point, at constant temperature through the latent heat transition, and below the freezing point. The theoretical quantity of heat to be removed can be readily calculated by various available methods. However the three-dimensional estimate of the thermal processes, considering the variation of the thermal properties of the medium associated with phase changes of water, is complex. Refined computations using FEM-techniques are under development. The number and spacing of freeze-tubes must take into account several variables, such as refrigeration capacity, thickness of the frozen zone, drilling conditions and costs. Choice of refrigerants should take into account the needs for thermal balance, as well as careful evaluations of operational, safety and environmental effects.

4.4 Summary

Design issues regarding grouting and ground freezing have been discussed in this chapter, using the data presented in the descriptive model as a basis. The conclusions can be summarised as follows:

- The reported post-grouting water inflows for the sections where NE-1 was encountered were approximately 50 l/min/10 m. The analysis indicates that the achieved hydraulic conductivity of the grouted rock in NE-1 is likely to be in the range of 5 to 8×10^{-8} m/s.
- In order to reduce the water inflow to 50 l/min/10 m for the conditions used in this study, the required hydraulic conductivity after grouting is likely to be in the range of 2 to 8×10^{-8} s/m, depending on the tunnel depth. These values of hydraulic conductivity can be achieved using current grouting technology.

- Regardless the differences in the initial values of the hydraulic conductivity between the transition zone and the core, the final hydraulic conductivity achieved in the grouted zone plays a determinant role for the amount of water inflow into the tunnel. This suggests that the required hydraulic conductivity specified for grouting performance could be the same for both the transition zone as well as the core of the fracture zone. However, the fact that the initial hydraulic conductivity is higher in the core zone than the transition zone means that the grouting process in the core zone will need to be more extensive than in the transition zone.
- Ground freezing is a possible alternative to grouting when considering water inflow control. The choice of ground freezing must, however, be carefully analysed and the risks, costs and time schedules must be taken into account. The thermal design must ensure that the freezing capacity must be sufficient with consideration of groundwater flows and salt content in the groundwater, which are the two major hindrances with regard to the freezing process. Monitoring during the entire freezing process is essential in order to ensure that the frozen zone performs according to the design.

5 Analysis of stability issues

5.1 General

The aim of this chapter is to address the stability related issues that were specified in Chapter 3, namely:

1. Stability associated with large deformations due to overstress of the rock mass.
2. Stability of the tunnel face.
3. Structurally controlled instability.

Specific stability issues that are related to the grouting and ground freezing processes are also considered in this chapter.

5.2 Overstressed rock

As discussed in Chapter 3, large deformations may occur in tunnels excavated in heavily fractured rock masses subjected to high initial rock stresses (in relation to rock mass strength). These deformations occur due to general shear failures in the rock mass caused by the stress field. The ground conditions resulting in time-dependent deformations associated with overstressing of the rock mass are commonly known as “squeezing ground”. Tunnelling in squeezing ground presents a variety of problems. Several case histories in which squeezing ground was encountered are described in Appendix 2.

Calculation of the predicted tunnel deformation in a weak rock mass subjected to high initial stresses is generally done using either numerical simulations (where initial stresses, rock mass behaviours and tunnel shapes can be modelled in detail) or analytical solutions e.g. /Hoek, 1980/ and /Stille, 1984/, where a general tunnel behaviour under elastic and elasto-plastic conditions can be analysed.

For this study the analytical solution developed by /Stille, 1984/ is to be used for the estimation of the tunnel deformations. The analytical solution includes inherent assumptions, however, it is felt to be reasonable for the purposes of this study. The use of an analytical solution can be justified by its simplicity and the fact that highly fractured rock can often be treated as a continuum. The solution uses the Mohr-Coulomb failure criteria and a non-associated flow rule, assuming a circular tunnel in a homogenous rock mass, subjected to a hydrostatic initial stress field. The analysis is performed by conducting a Monte-Carlo simulation using the computer program Crystal Ball in order to obtain a probability distribution of the tunnel deformations under different conditions.

/Anagnostou and Kovári, 2003/ suggest that the rock mass on either side of a fracture zone can provide certain favourable supporting effects (wall stabilising effect) due to the shear stresses τ that occur at the host rock/fracture zone interface (Figure 5-1).

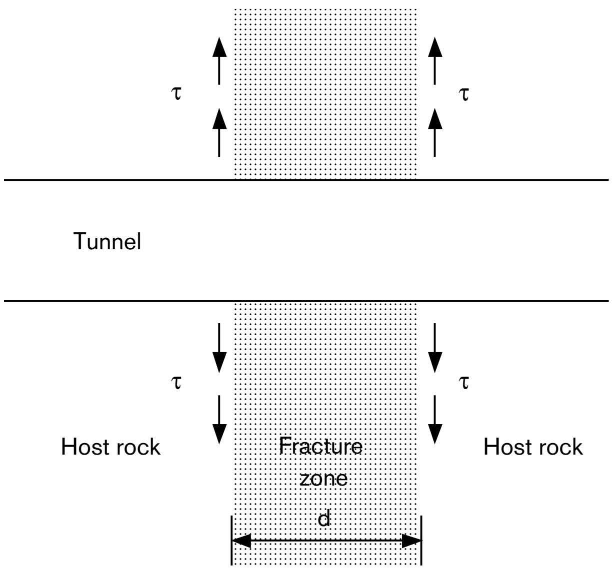


__Figure 5-1.__ Supporting effect of the host rock (wall stabilising effect) to a fracture zone /Anagnostou et al. 2003/.

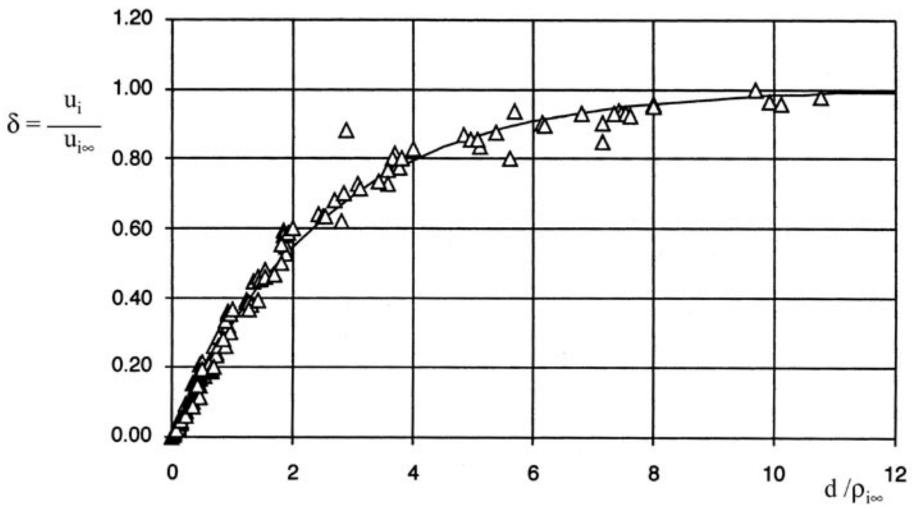


__Figure 5-2.__ Deformation reduction factor δ as a function of d/ρ∞ /Anagnostou et al. 2000/.

/Anagnostou and Kovári, 2003/ have conducted three-dimensional finite element modelling to study the supporting effects of rock masses surrounding fracture zones. The following relationship has been proposed based on the results of these analyses, (see also Figure 5-2):

$$\delta = \frac{u_i}{u_{i\infty}} = 1 - e^{-0.4\frac{d}{\rho_\infty}} \qquad (5\text{-}1)$$

where:

d = width of fracture zone;

δ = deformation reduction factor;

u_i = tunnel deformation in the fracture zone with width d;

$u_{i\infty}$ = tunnel deformation without consideration of the wall stabilising effect;

ρ_∞ = the radius of the plastic zone without consideration of the wall stabilising effect.

The radius of the plastic zone denoted as ρ_∞ can be interpreted as an index showing the degree of plasticity to which the rock mass is subjected. The plasticity exhibited in a rock mass around a tunnel is the major cause of the tunnel deformation. Theoretically the radius of the plastic zone can be determined by using the existing analytical solutions. For this study, the deformation reduction factor δ expressed in equation (5-1) is incorporated into the analytical solution and hence consideration of the supporting effect of the surrounding rock is included in the analysis.

The rock mass properties used in the analysis are those proposed for the descriptive model (see Chapter 2) and are given in Table 5-1 for the transition zone and Table 5-2 for the core of the fracture zone. The dilatancy factor describing the volumetric expansion of rock during yielding is assumed to be 1.4, which corresponds to a dilatancy angle of about 10°. The initial rock stresses used for the three depths are given in Table 5-3. The tunnel radius has been set to 4 m (giving a tunnel area of 48 m^2). The variable parameters in the Monte-Carlo simulation are the modulus of elasticity, cohesion, friction angle and initial stresses. The probability density functions (PDF's) for the initial stresses are assumed to be uniform. Triangular distributions with assumed mean values have been used for all the other remaining parameters.

Table 5-1. Rock mass properties of the transition zone based on the descriptive model for this study.

	Min	Mean	Max	PDF
E-modulus. GPa	5.6	11.7	17.8	Triangle
Cohesion. MPa	0.8	1.2	1.6	Triangle
Friction angle. deg	30	35	40	Triangle
Dilatancy factor	1.4	1.4	1.4	–
Poisson's ratio	0.25	0.25	0.25	–
Equivalent σ_{cm}. MPa	2.8	4.6	6.9	–

Table 5-2. Rock mass properties of the core of fracture zone based on the descriptive model for this study.

	Min	Mean	Max	PDF
E-modulus. GPa	1.9	3.8	5.6	Triangle
Cohesion. MPa	0.6	0.7	0.8	Triangle
Friction angle. deg	20	25	30	Triangle
Dilatancy factor	1.4	1.4	1.4	–
Poisson's ratio	0.25	0.25	0.25	–
Equivalent σ_{cm}. MPa	1.7	2.2	2.8	–

Table 5-3. Initial stresses (MPa) used in the analysis.

Depth	Min	Max	PDF
200 m	5	16	Uniform
400 m	10	29	Uniform
600 m	14	43	Uniform

The results of the analysis for the transition zone and core zone are presented as cumulative frequency diagrams in Figure 5-3 and Figure 5-4 respectively. The results are expressed in terms of "relative strain", defined as the ratio between the radial deformation u_i and the tunnel radius R_i. The calculated mean values of the relative strain and corresponding displacements are presented in Table 5-4. It should be noted that rock supports are not included in the calculations.

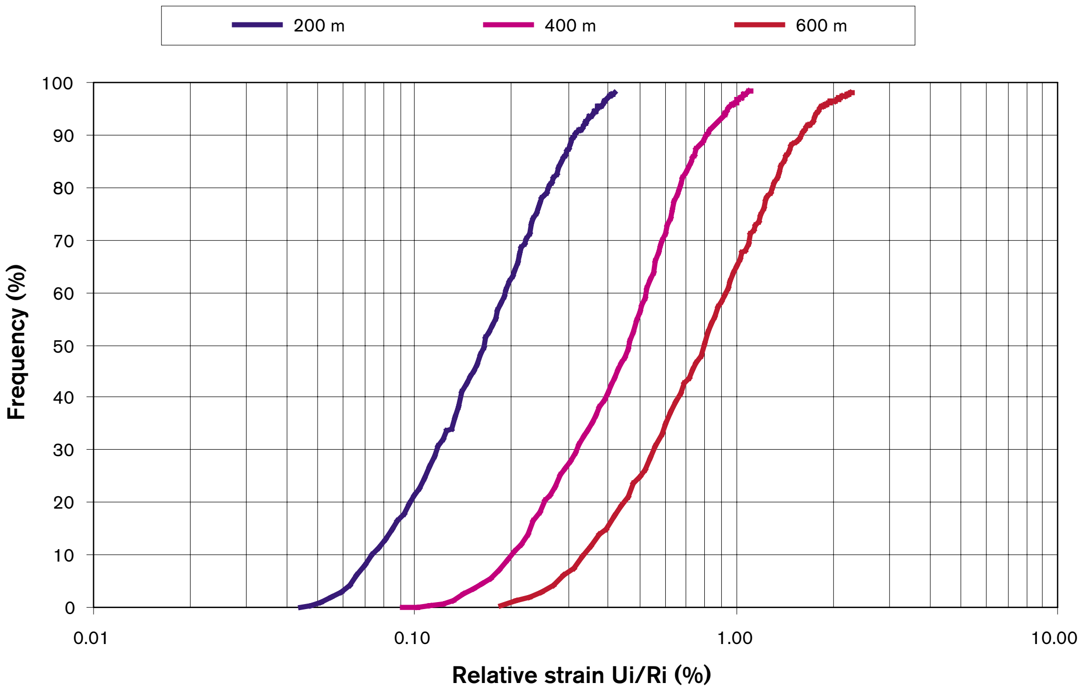


Figure 5-3. *Calculated cumulative frequency of relative strain for transition zone with consideration of the supporting effects of the host rock.*

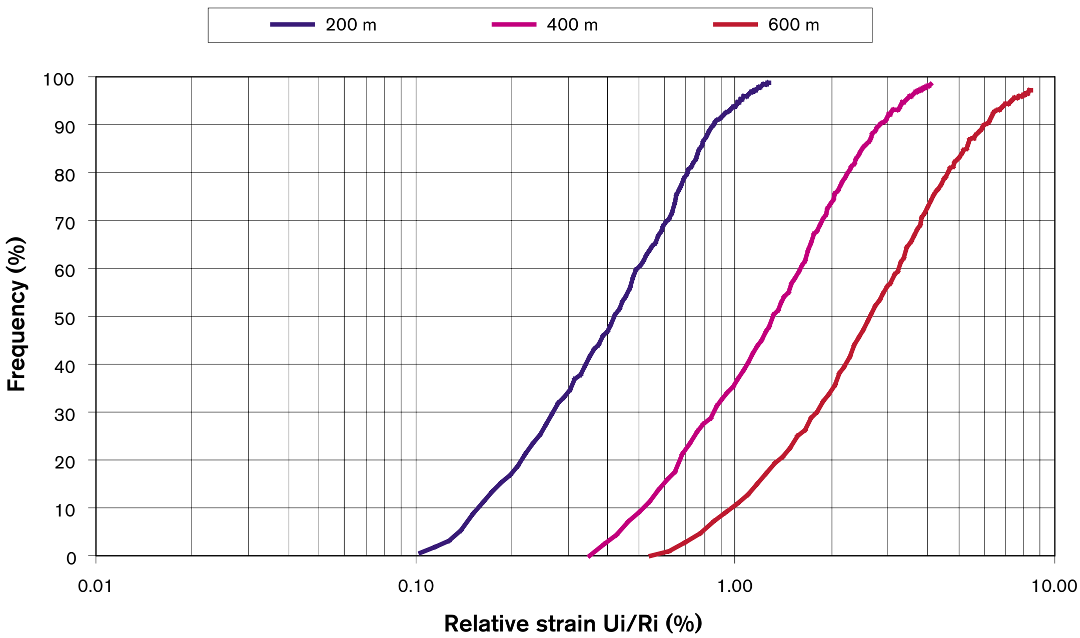


Figure 5-4. *Calculated cumulative frequency of relative strain for the core of the fracture zone with consideration of the supporting effects of the host rock.*

Table 5-4. Calculated mean values of relative strains u_i/R_i and displacements (with consideration of the supporting effects of the host rock).

Depth	Transition zone	Core of fracture zone
200 m	0.2% (8 mm)	0.5% (20 mm)
400 m	0.5% (20 mm)	1.5% (60 mm)
600 m	0.9% (36 mm)	3.2% (128 mm)

The calculated mean values of relative strain for the transition zone shown in Table 5-4 are relatively small for all depths. The corresponding mean values of deformation in the transition zone will be approximately 10, 20 and 40 mm at 200, 400 and 600 m depth respectively. In contrast, the mean values in the core zone will be approximately 20, 60 and 130 mm at 200, 400 and 600 m depth respectively. The deformations in the core zone at 400 and 600 m depths are considerably higher than the other cases. It is also worth noting that the variance of the calculated relative strains for the core of the fracture zone is larger than that for the transition zone, which indicates that a larger uncertainty would be expected for the deformations in the core zone.

The magnitude of these relative strains can be compared with criteria proposed by /Hoek and Marinos, 2000/ which indicate the problems that are likely to occur due to deformation of the rock mass. This criterion is essentially proposed for problems associated with tunnelling through squeezing ground. A modified criterion that is felt to be more applicable to the problems outlined for this study is given Table 5-5.

Table 5-5. A criterion for estimating the degree of difficulty associated with tunnelling through overstressed rock (Note that the relative strain is for tunnels with no support) /modified after Hoek and Marinos, 2000/.

Difficulty class	Relative strain %	Geotechnical issues	Support types
A	Less than 1	<u>Very simple tunnelling conditions, few stability problems</u>. Tunnel design recommendations based upon rock mass classifications provide an adequate basis.	Rockbolts and shotcrete typically used for support.
B	1 to 2.5	<u>Minor tunnelling problems</u>. Ground Reaction Curve methods are used to predict the formation of a plastic zone surrounding a tunnel and of interactions between the progressive development of this zone and different types of support.	Tunnel support with rockbolts and shotcrete; sometimes light steel sets or lattice girders are added for additional security.
C	2.5 to 5	<u>Problematic tunnelling conditions. Face stability is generally not a major problem</u>. Finite element analyses, incorporating support elements and excavation sequence are normally performed.	Rapid installation of support; careful control of construction quality. Heavy steel sets embedded in shotcrete are generally required.
D	5 to 10	<u>Severe tunnelling and face stability problems</u>. Finite element analyses are generally carried out. Some estimates of the effects of forepoling and face reinforcement are required.	Forepoling and face reinforcement are usually necessary.
E	More than 10	<u>Very severe tunnelling and face stability problems</u>. No effective design methods are currently available. Most solutions are based on experience.	Forepoling and face reinforcement are usually applied and yielding support may be required in extreme cases.

The following points can be drawn by comparing the results presented in Figure 5-3 and Figure 5-4 with the criteria proposed in Table 5-5:

1. Stability problems due to overstressed rock are unlikely to occur when tunnelling through the transition zone at depths of 200 m and 400 m (difficulty class A). As depth increases to 600 m, there is a 70% chance of the relative strain falling in difficulty class A and 30% for class B.
2. For the core zone, a summary of the probability of the different difficulty classes being encountered at the three different depths is presented in Table 5-6. The table shows that there is only a 5% chance of encountering minor tunnelling problems (class B) at 200 m depth. As the depth of excavation increases to 400 m, the potential for encountering problems due to overstressed ground increases. This is emphasised by the fact that there is a 60% chance that minor problems will be encountered (class B) and a 5% chance that problematic tunnelling conditions will occur (class C). According to Table 5-5 these types of problems should be controllable using rockbolts and shotcrete with the possible introduction of steel sets. At 600 m depth, the severity of the problems increases due to the increase in stress and there is a 15% chance of encountering severe tunnelling and face stability problems.

The review of the previous reports and documentation supplied by SKB did not find any reports of instability encountered during the passage of deformation zone NE-1 at approximately 200 m. This is in broad agreement with the findings of the stability analyses presented above.

Table 5-6. Estimated probability of difficulty class for the core of the fracture zone with consideration of the wall stabilizing effect.

Depth	Expected difficulty class				
	A	B	C	D	E
200 m	95%	5%			
400 m	35%	60%	5%		
600 m	10%	40%	35%	15%	

5.3 Stability issues associated with grouting

The following two issues associated with grouting are to be discussed in this section:

- Stability of the grouted zone;
- Possible improvement of rock mass properties due to grouting.

The fact that the grouted zone has a lower permeability than the surrounding rock mass means that it will be subjected to water pressure acting on its outer boundary. The grouted zone would be subjected to the original water head if it is assumed to be completely impermeable. In reality, a grouted zone around a tunnel is not completely impermeable. Therefore, the water pressure acting on the grouted zone is somewhat lower than the original water head, and is dependent on the pattern of water seepage through the grouted zone. The grouted zone must, therefore, be designed to have a sufficient thickness to be able to sustain the water pressure acting on its outer boundary.

In addition, a hydraulic gradient (difference in water pressure over a short distance) associated with the water seepage will exist within the grouted zone. This hydraulic gradient must be taken into account when considering the equilibrium of the stresses in the rock mass. General equilibrium equations, which consider hydraulic gradients, can be found in textbooks for the theory of pore-elasticity. Assuming a simplified case as illustrated in Figure 5-5, where a circular tunnel in a porous medium is subjected to a hydrostatic water pressure, the equilibrium of the thin ring results in the following equilibrium equation:

$$\sigma_{tw} = r \cdot \frac{dp_w}{dr} \tag{5-2}$$

Where, r is the radius of the ring, dp_w/dr is the hydraulic gradient and σ_{tw} is the tangential stress induced by the hydraulic gradient. It is worth noting that the stress σ_{tw} induced by the hydraulic gradient is an additional stress to that caused by the initial rock stress.

The stress induced by the hydraulic gradient may become significant when the hydraulic gradient in a grouted zone is high. For instance if an 8 m diameter circular tunnel with a 10 m thick grouted zone is subjected to a water pressure of 6 MPa (equivalent to 600 m depth under ground water level), the average hydraulic gradient inside of the grouted zone will be 0.6 MPa/m. This will result in an tangential stress of about 2.5 MPa at a point close to the tunnel surface by using equation (5-2). Theoretically speaking, the hydraulic gradient is highest at the points closest to the tunnel surface /Anagnostou et al. 2003/. This means that the tangential stress in the grouted zone close to the tunnel surface will be the greatest.

The stability of a grouted zone with consideration of the stresses induced by the hydraulic gradient has yet to be studied in depth. It can be theoretically hypothesised that a high hydraulic gradient could result in high stresses in a grouted zone. These stresses together with the stresses induced by the initial rock stresses would cause plasticity (shear failure) in the rock mass, which would consequently deteriorate the integrity of the grouted rock mass. As a result, the water seepage would probably be increased resulting in a redistribution of the hydraulic gradient. This chained interaction between the stresses and hydraulic gradient could lead to deteriorated stability conditions of the grouted zone. For tunnels under high water pressure at great depths, this issue becomes more significant and it is strongly recommended to be studied in detail. Such studies can be undertaken by means of numerical analysis and physical model testing.

Grouting is commonly seen as a method for sealing a tunnel to prevent the ingress of water. However, there is evidence to suggest that in fractured rock conditions grouting may also improve the strength of the rock mass. The effects of the strengthening effect of grouting have been reported by several authors, including /Barton et al. 2001; Kikuchi et al. 1999/ and /Roald et al. 2003/.

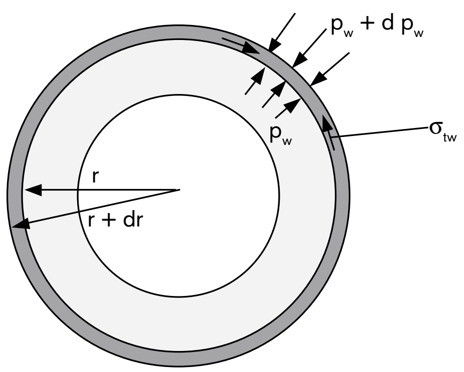


Figure 5-5. *The stress induced by the water pressure gradient around a circular tunnel.*

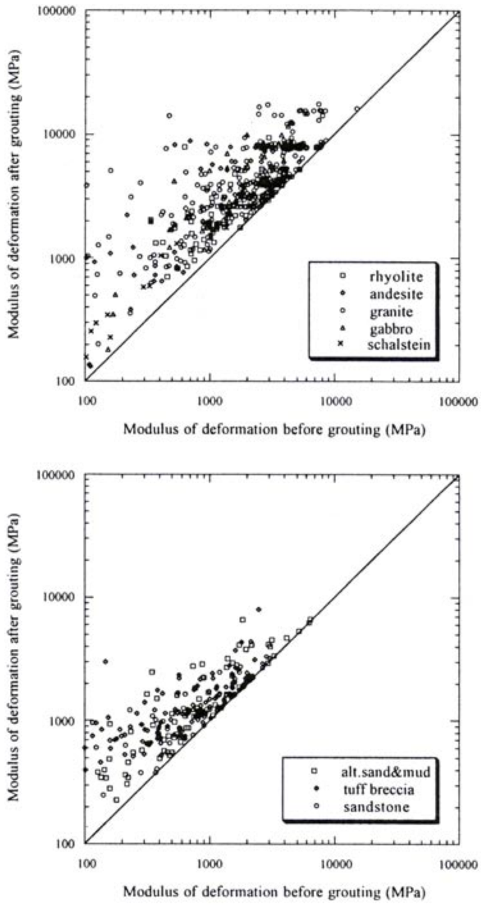


Figure 5-6. *Improvement of the modulus of deformation due to grouting /Kikuchi, 1999/.*

Figure 5-6 shows the effects of grouting on the modulus of deformation, based on the results from borehole expansion tests conducted by /Kikuchi et al. 1999/. The results show that the modulus of deformation can be significantly improved by grouting (on average by a factor of about 2.0). It can also been seen that the improvement in the modulus of deformation due to grouting is greater for poorer quality rock than it is for good quality rock. Since there may be correlation between rock mass strength and the modulus of deformation, it can be hypothesised that grouting can also have a positive impact on rock mass strength.

An improvement in the rock mass properties within the grouted zone around a tunnel would theoretically result in a reduction in the amount of deformation occurring in the rock mass. A closed-form analytical solution for a grouted circular tunnel which considers the elasto-plastic behaviour of the rock mass in both the grouted and non-grouted zone was developed by /Chang and Hässler, 1993/. This method has been used in this study to make a preliminary assessment. The results suggest that a 10% improvement in the modulus of elasticity, cohesion and friction angle will decrease the relative strain by a factor of approximately 0.6 for an unsupported tunnel.

The improvement in rock mass properties due to grouting, particularly in highly fractured rock masses, is a topic that is still relatively un-researched. It is therefore suggested that the positive effects of ground strengthening due to grouting are used as an additional safety margin.

5.4 Stability associated with ground freezing

Though the primary purpose of freezing in this project is to control water inflow, design of the ground freezing must ensure that tunnel stability is maintained during and after freezing. The design should satisfy:

- Short-term stability during ground freezing, which is dependent on the load bearing capacity of the frozen zone and the necessary temporary rock support;
- Long-term stability after tunnel excavation, which is dependent on the load bearing capacity of the permanent rock support.

The major issue for the short-term stability is to determine the thickness of the frozen zone, so that the frozen zone has sufficient bearing capacity. Because the frozen zone is impermeable, it will be subjected to the water pressure equal to the initial water head. In addition, a load from the surrounding rock will act on the frozen zone due to the tunnel excavation. Therefore, the following factors must be taken into account when determining the required thickness of the frozen zone:

- The static water pressure denoted p_w ;
- The load from the surrounding rock, i.e. the ground pressure, denoted as p_g;
- The strength and stiffness of the frozen zone;
- The time-dependent characteristics of the frozen zone.

Theoretically, the frozen zone may have one of the following conditions:

- An elastic condition.
- An elasto-plastic condition.
- A completely plastic condition.

Past experience indicates that it is unnecessarily conservative to design the frozen zone using an elastic condition /Harris, 1995/. On the other hand, a frozen zone subjected to a completely plastic condition would result in large deformations (or shear deformation) in the rock mass, which could result in failure in the integrity of the frozen zone. The failure could occur along the whole length of the fracture if large shear deformations occur. High-pressure water could therefore penetrate through the fractures and, in the worst case water might stream through the whole frozen zone (Figure 5-7). If the leakage cannot be plugged due to e.g. a lack of freezing capacity, the total stability of the entire tunnel would be under threat. Therefore, block movements and deformations in the frozen zone caused by tunnel excavation are important factors to be taken into account when conducting the design of the frozen zone.

To ensure safety during tunnel excavation through the frozen zone, temporary rock supports are likely to be necessary. Installation of any rock bolts must be done with great care to avoid damaging the freezing-tubes. As an alternative, steel sets and/or spilling installed at the same time as the installation of freezing-tubes may be a more suitable alternative to rock bolts. Shotcrete has also been used as temporary support for frozen ground in tunnels however, the effects of the cold rock surface must be carefully evaluated and tested. /Harvey, 1993/ reported that tests on cores from shotcrete linings in a frozen tunnel, compared with those on non-frozen cores showed no adverse effects. The freezing technique has been used in tunnel projects in Stockholm /Stille et al. 2000/, showing that the strength of the cores of the shotcrete lining after 4 days was in the range 33–35 MPa.

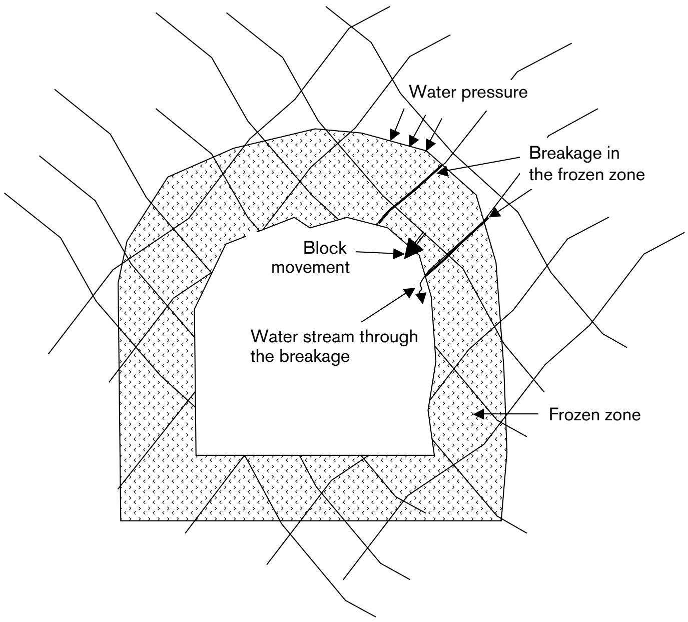


Figure 5-7. *Water stream through breakage in the frozen zone due to block movement.*

Design of the thickness of the frozen zone and the temporary support is generally carried out by means of numerical analyses. Ground Reaction Curves (GRC), as illustrated in Figure 5-8, may also be useful for evaluating the basic mechanical behaviour of a frozen zone. It can be seen that a frozen zone with elastic response because of e.g. larger thickness will result in a higher load on the frozen zone. This means that a frozen zone with larger thickness will not necessarily lead to higher factor of safety, but higher costs.

The most common method of permanent support associated with ground freezing is a concrete lining. Steel linings may also be considered if a concrete lining is prohibitively thick. In the design of such steel permanent linings, fire and corrosion protections are of special concern.

Permanent concrete linings should be designed to bear both the static water pressure as well as the ground pressure. Because a cast-in-place concrete lining has very low permeability, it will be subjected to a water pressure equivalent to the initial water head.

To determine the final ground pressure that will be exerted on the permanent lining, the interaction between the lining and the rock mass during the thawing process must be taken into account (Figure 5-9). Previous studies indicate that the freezing and thawing process may have an influence on the properties of the rock mass. For instance, the aperture of an existing fracture may be increased due to the volume expansion when ice crystals form. As a result, the increased aperture will lead to a reduction in the contact between the fracture walls and hence result in a reduction in the strength of the fracture. This may be one of the reasons why a rock mass can exhibit time-dependent behaviour during and even after the thawing period.

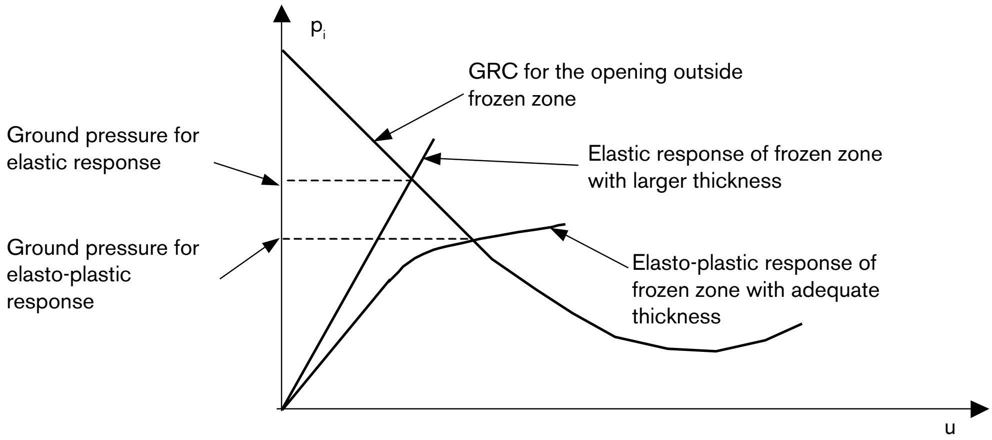


Figure 5-8. *Elementary mechanical behaviour of a frozen zone.*

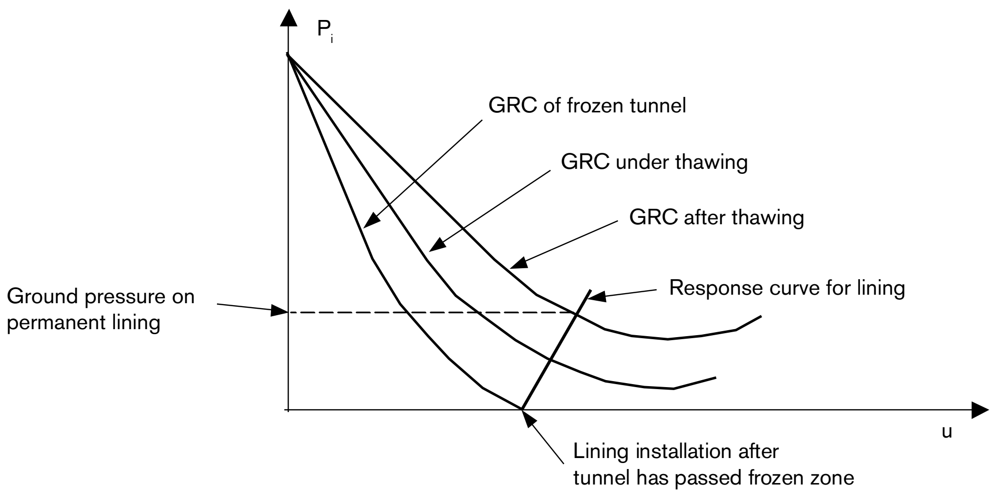


Figure 5-9. *Interaction between the tunnel and the lining during the thawing process.*

To estimate the required thickness of a concrete lining, a simplified method using equation 5-3 is proposed for this study.

$$T = R_i * (P_w + P_g)/\sigma_c \qquad (5\text{-}3)$$

Where:

T = Thickness of the lining (m)

R_i = Excavated tunnel radius (m)

P_w = Static water pressure (MPa); P_g = ground pressure (MPa)

σ_c = Compressive strength of lining (MPa).

Although the formula is essentially for thin-wall-problems, it is a useful method for obtaining an estimate of the required lining thickness.

Assuming that a concrete lining has a compressive strength of 25 MPa and is subjected to a water pressure of 6 MPa (equivalent to full hydrostatic pressure at 600 m depth) and a ground pressure of 1 MPa, the required thickness of the lining according to equation 5-3 is approximately 2 m. The calculation takes into consideration the fact that a larger diameter tunnel must be excavated to provide space for the lining (i.e. the excavated tunnel radius will be increased from 4 m to 6 m). This increase in the tunnels dimensions should be carefully evaluated in further studies as it may have a significant impact on tunnel construction.

5.5 Stability of tunnel face

Instability of the tunnel face can be categorized into the following:

- Sudden face failure caused by high water pressure;
- Flowing ground;
- Fallout of blocks (blocks pushed out by water pressure);
- Large tunnel face deformations.

The distance of the tunnel face to the fracture zone plays an important role when considering tunnel face stability. Water pressure acting on the rock mass in front of the face can lead to failure if the distance between the water-bearing structure and the tunnel is too small (Figure 5-10). Analysis of the response of the rock mass between the tunnel face and the fracture zone for various distances using numerical modelling should enable to determine the safe distance.

Even if the tunnel face is at a sufficiently safe distance from the water-bearing zone to prevent the type of failure highlighted in Figure 5-10, water may still be introduced into the fractures closer to the face through the boreholes drilled for probing, grouting or freezing purposes (Figure 5-11). If this occurs the "effective distance" between the face and the surface where the water pressure is acting will be reduced. This may also lead to failure of the tunnel face in form of e.g. block pushed out by the water pressure . Steel standpipes should be installed along the entire length of each borehole that penetrates the water-bearing zone, in order to prevent the water from entering the fractures in front of the face via boreholes. The steel standpipe is commonly combined with a so-called "blow-out prevention" system for drilling operation under high water pressure (see Chapter 6).

If the rock mass in front of the tunnel face has a sufficiently high permeability, erosion of the fill material within the fractures may occur due to the large volumes of water flowing through the fracture network. This inner erosion of the fractures will in turn increase the water seepage, which could lead to a progressive failure of the face. It is, therefore of vital importance that the rock mass in front of the face is grouted.

The fact that grout dispersal seldom results in a uniform distribution of cement means that there is a potential risk that the grouted zone may, in places, have a smaller extent than expected. This could potentially lead to large water inflows in the tunnel which, if combined with loose material from the fracture zone can result in flowing ground when the face approaches the fracture zone (Figure 5-12).

It is worth noting that most cases of tunnel face failure due to water pressure are because of unsatisfactory probing, that may result in a water-bearing zone being encountered unexpectedly. In such a case there is seldom sufficient time to implement remedial measures.

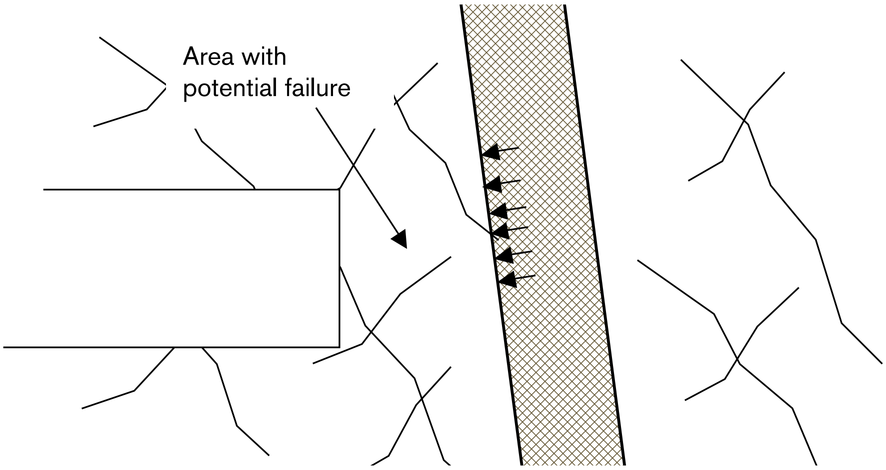


Figure 5-10. *Area of potential failure between the water-bearing fracture zone and the tunnel face.*

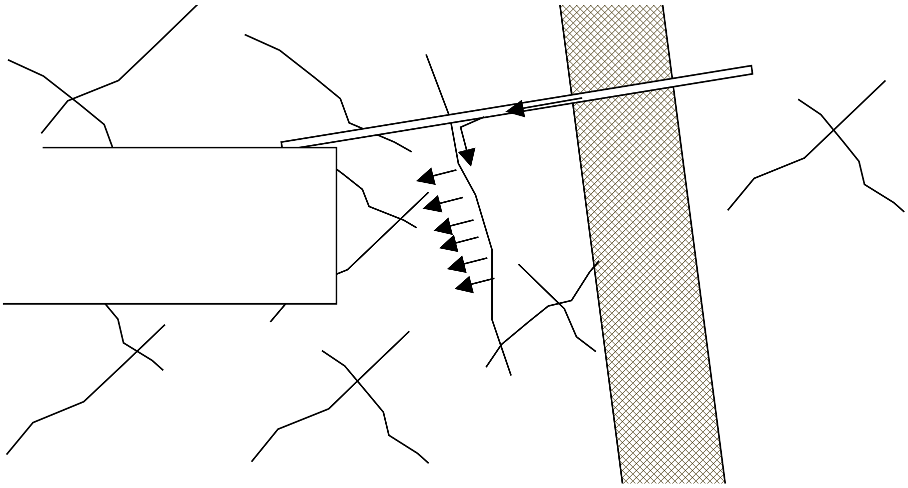


Figure 5-11. *Water is introduced into the fractures closer to the face via boreholes.*

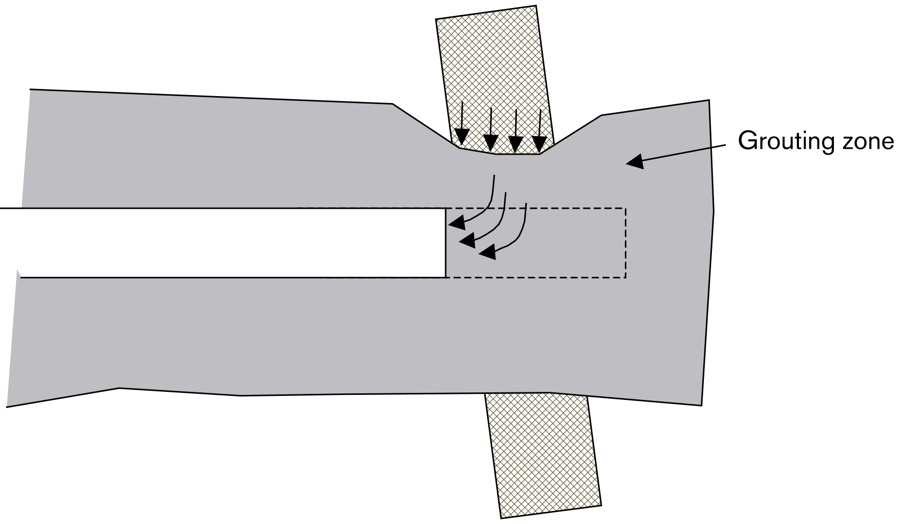


Figure 5-12. *Misjudgement of the extension of the grouted zone may cause flowing ground.*

An effective measure for preventing the face from collapse due to high water pressures is observation of water inflows at the face. Large water leaking from fractures at the face is an early indication of the presence of water behind the face. Examination of the tunnel face for water seeping from fractures should therefore be emphasized during tunnelling operations.

Behaviour of the tunnel face in highly fractured rock conditions has been studied by numerous authors, including /Lombardi, 1979; Panet, 1982/ and /Chang, 1994/. The studies show that a significant plastic zone can occur in the vicinity of the tunnel face when a tunnel is subjected to a high stress level (see Figure 5-13). The extension of the plastic zone is obviously dependent on the rock mass strength, initial rock stresses and the shape of the tunnel. Such plasticity of the face could lead to collapse of the face, as observed in tunnels that have been driven in poor rock conditions using inadequate support /Hoek, 2000/. There are various measures to promote face stability, such as spilling, bolting of the face, installation of dowells or shotcreting.

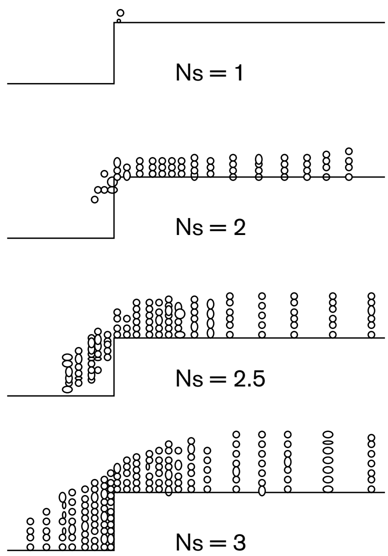


Figure 5-13. *Plasticity in the vicinity of tunnel face /Panet, 1982/ (Ns = σ_o/c_u, where σ_o = initial stress and c_u = cohesion of rock mass).*

5.6 Structurally controlled instability

Structurally controlled instability may occur when wedges or blocks, driven by gravity and other loads (e.g. water pressure), overcome the total strength mobilised along fracture planes. The support of these wedges is usually achieved by the installation of rock bolts and shotcrete, as illustrated in Figure 5-14. The load bearing mechanism of these types of support has been studied by numerous authors, including /Holmgren, 1985; Opsahl, 1982/ and /Nilsson, 2003/. Figure 5-14 shows that in addition to the shear resistance of the fracture planes and possible supporting effects of the stresses in the rock mass, the bearing capacity of the support system consists of the following components:

- Support of the bolts.
- Shear strength of the shotcrete lining.
- Adhesion of the shotcrete lining to the rock surface.

There are various available methods for the design of rock supports in jointed rock masses. The most common practice in Sweden is that the rock support design is verified and finalized after geological mapping has been conducted at the tunnel face. Based on the actual data of the fractures, potentially movable wedges are identified and the weight of each wedge is then calculated. With consideration of the loads and supporting components (as shown in Figure 5-14), the final required bearing capacity of the support is assessed. The software Unwedge combined with other means is often used for the assessment of the rock support.

The effects of the all the components, shown in Figure 5-14 must be considered in the detailed design of rock support for structurally controlled failure. For the purposes of this study, a preliminary estimation of the weight of potentially unstable blocks has been undertaken. As mentioned above, by knowing the weight of a potentially unstable block, required support can be estimated. An approach termed “Method for Simulation of Blocks” (MSB) developed by /Jakubowski, 1994/ has been used. The MSB-approach is a probabilistic extension of the Key Block Theory /Shi et al. l981/, which identifies unstable blocks around a tunnel by conducting Monte-Carlo simulations. Details of the method are given in Appendix 3.

The geological conditions in the transition zone are considered as being more susceptible to block failure than the core, due to the blocky nature of the rock mass. Several fracture sets have been identified for the rock mass around NE-1, and these have been presented in the descriptive model. Three of these fracture sets, namely J1, JW1 and JW2 are interpreted to be likely to promote structurally controlled instability in the transition zone. Fracture sets JW1 and JW2 were mapped in the tunnel as being water-bearing fractures, while J1 is one of the most frequently occurring fracture sets encountered in deformation zone NE-1. Table 5-7 is a presentation of the estimated weights of large unstable blocks over a 35 m long stretch of tunnel. A friction angle of 30° and a cohesion of 0 MPa for the fractures are used in the calculations for an unsupported tunnel. The direction of the access tunnel for Äspö HRL is used in the calculation. Because the aim of the calculation is to estimate the weights of potentially unstable blocks, water pressure and rock stresses are not included.

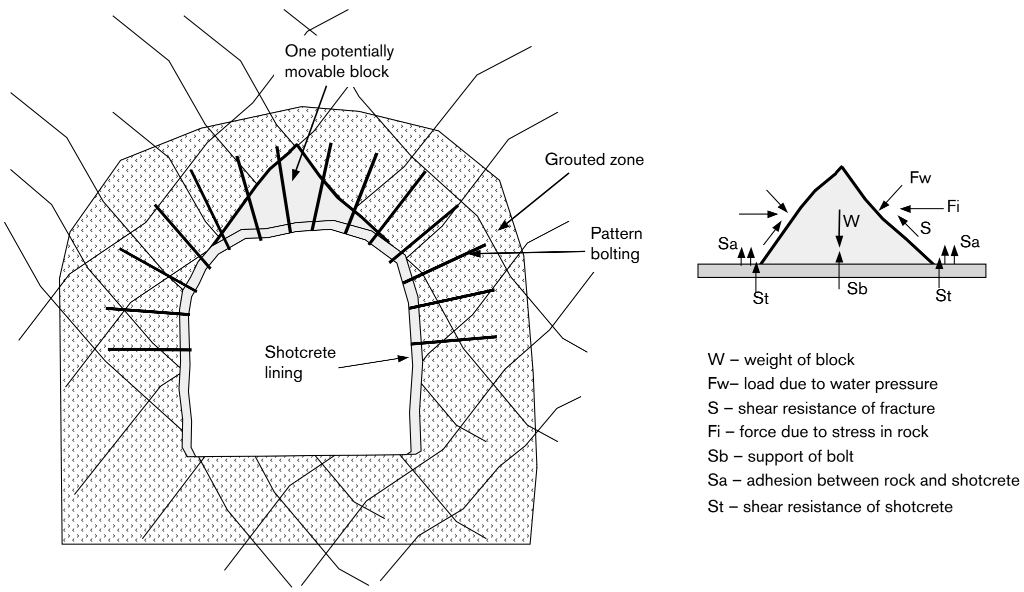


Figure 5-14. *Supports of fractured rock to prevent fall of potential unstable blocks.*

Table 5-7. Estimated weights of potentially unstable blocks.

Volume of unstable block (m^3)	Weight of unstable block (kN)	Mean number of unstable blocks per 35 m tunnel length
2 < = Vol < 5	52–130	24
5 < = Vol < 10	130–260	8
10 < = Vol < 20	260–520	2
20 < = Vol	520 <	< 1

It can be seen from Table 5-7 that most of the potentially unstable blocks are smaller than 5 m^3 in volume and weigh less than 130 kN. Even though only the friction of the fractures is considered in the calculations, the results suggest that unstable blocks larger than 20 m^3 or weighing in excess of 520 kN are unlikely to occur. A block with a weight of 520 kN can be secured by approximately 5 rock bolts with an allowable capacity of 120 kN each. It can therefore be concluded that structurally controlled instability is not likely to be a major concern, but must be considered in the rock support design.

5.7 Summary

The following stability issues have been discussed in this chapter:

- Stability associated with large deformations due to overstress of the rock mass;
- Stability of grouted zone;
- Stability of frozen zone;
- Stability of the tunnel face;
- Structurally controlled instability.

Large deformations due to overstressing of the rock mass in the transition zone are unlikely to occur, even at 600 m depth. The reduction in rock mass quality in the core zone however, means that large deformations are likely to result in tunnelling difficulties at 400 m and particularly 600 m depth. The estimated mean values of deformation for an unsupported tunnel in the core are 60 mm and 130 mm at 400 and 600 m depth respectively. The stability problems associated with such deformations can essentially be dealt with using conventional support methods in form of rockbolts and shotcrete, however, the introduction of steel sets and the use of a forepole umbrella may be required.

The thickness of the grouted zone must be sufficiently large in order to sustain the water pressure acting on its outer boundary. Moreover, the stresses induced by the hydraulic gradient inside of the grouted zone must be carefully evaluated. The latter issue has yet to be studied in detail and further research in this field is recommended.

If ground freezing is to be used, stability of the frozen zone must be considered. The thickness of the impermeable frozen zone is of importance, as it must be sufficient to bear the water pressure equivalent to the initial water head. Design of the permanent lining to ensure long-term stability must take into consideration both the ground pressure and the water pressure at the depth in question. Preliminary estimates indicate that the required thickness of the permanent concrete lining could be approximately 2 m at a depth of 600 m.

Stability of the tunnel face must be maintained throughout the tunnelling process and can be achieved by applying the following measures:

- Maintaining a safe distance between the tunnel face and the water-bearing zone during pre-grouting or freezing operations.
- Observation of the tunnel face.
- Carefully probing to determine the position of the water-bearing zone and to determine the water pressure in the zone;
- Face support, which may a combination of rock bolts, fibre glass dowels and shotcrete.

The analysis of the fracture sets from the descriptive model, which were deemed most likely to promote structurally controlled instability, indicates that block instability is not a major issue in the transition zone. In the design of the rock support, all acting forces including e.g. water pressure in the fractures must however be considered.

6 Construction issues

6.1 General

The objective of this chapter is to describe the major issues to be considered during tunnel construction, namely:

- Probing and drilling operation;
- Grouting arrangement;
- Execution of ground freezing;
- Rock supports associated with large deformations;
- Monitoring and back-analysis.

6.2 Probing and drilling

Probing is an essential part of modern tunnelling in order to minimize risks for unexpected rock conditions. Probing for water-bearing zones is even more important for tunnelling at great depths. Detailed probing programs must be set up with established reporting routines and action plans for unexpected occurrences. In tunnelling through a water-bearing fracture zone, the main objectives of probing are:

- To locate the water-bearing zone with higher precision;
- To investigate the geological as well as hydrogeological conditions of the zone;
- To provide data for determining the “entry point” for the passage of the zone;
- To continuously update the characteristics of the fracture zone.

Drilling operations through a water-bearing zone with high water pressure can be hazardous and troublesome. The following points are some of the important issues:

- Control of water flows from the boreholes during drilling operations;
- Drilling rate and precision for long boreholes to ensure the efficiency of the tunnelling operation;
- Feeding capacity and stability of the drilling rig to overcome the water pressure load.

Modern drilling techniques are available for handling problems related to high water pressures. It is advisable to hire contractors who specialize in drilling under these types of difficult conditions.

To prevent an uncontrolled large amount of water flushing out from boreholes, a so-called “blow-out-preventer” system is often installed to each borehole during drilling operations. An illustration of such a device is shown in Figure 6-1.

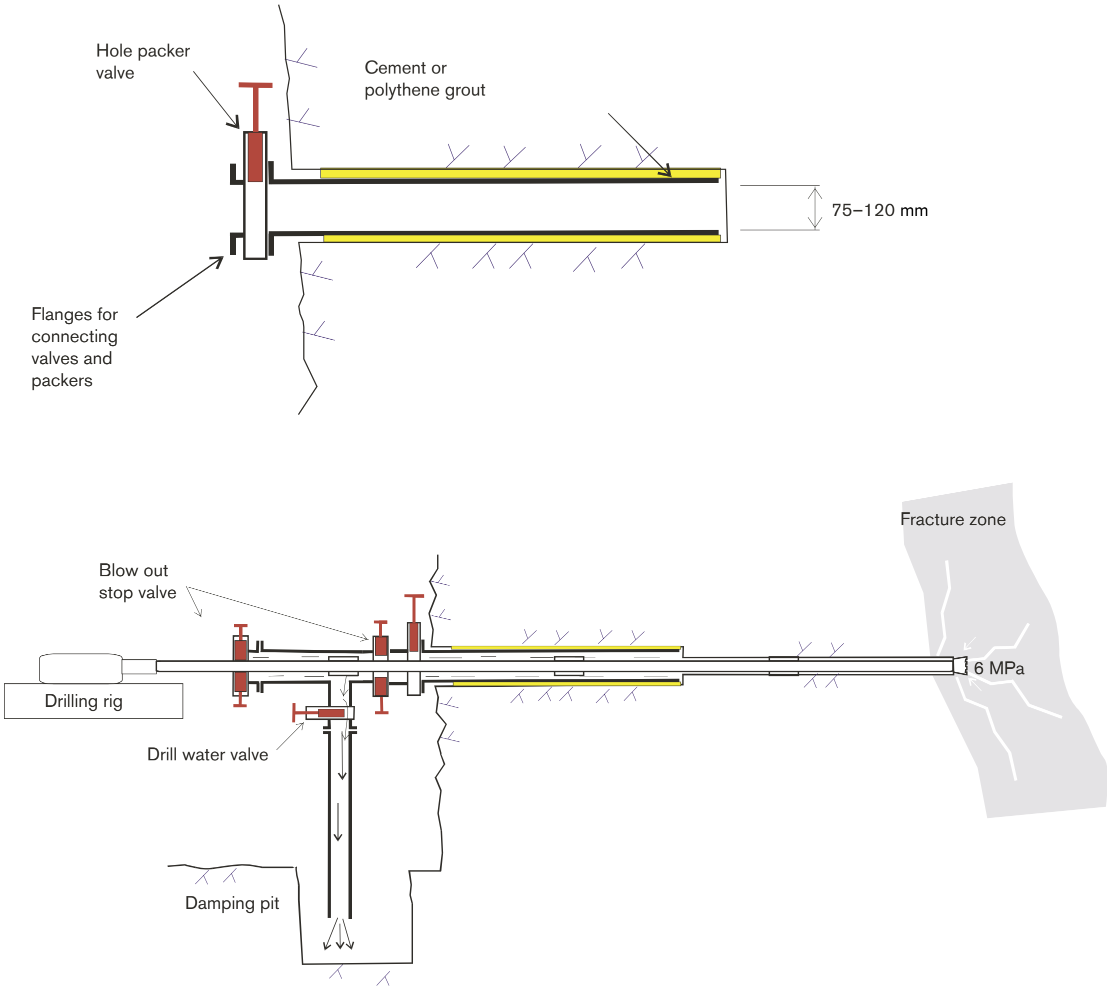


Figure 6-1. *Sketch illustrating a "blow-out-preventer" system.*

For the case concerned in this study, boreholes drilled for pre-grouting or freezing will have lengths of between 50–60 m. These holes must be drilled with an acceptable precision and drilling rate to achieve the expected objectives. Drilling with ordinary top-hammers has a high production rate, but the precision becomes poor when drilling distances exceeding about 20–30 m. For precision drilling, core drilling or DTH hammer-drilling may be necessary. Core drilling, even without core recovery, has a high drilling precision, but it is slow compared with top-hammer-drilling. Nevertheless, core drilling may be an alternative under high water pressure due to its precision and the lower flushing pressure.

The DTH technique results in larger holes (e.g. the smallest Atlas Copco DTH diameter is 85–90 mm) and this may be favourable for more efficient grouting of small fractures. One drawback, however, is that larger holes may result in greater volumes of water entering the tunnel.

6.3 Grouting arrangement

In Chapter 4, possible solutions for grouting of the water-bearing zone were proposed. Unexpected "wash-outs" may occur when grouting water-bearing zones under high water pressure. In such situations it is necessary to accelerate the cement setting and hardening times. Before using accelerators, site tests have to be undertaken to determine the open time, setting time and hardening time, with the actual equipment, type of cement and site temperature etc /Garshol, 2003/.

Due to the high water pressure and presence of fractured fragments in the core zone, large quantities of water mixed with rock segments and gravel might be flushed out from the borehole. The installation of traditional packers in such boreholes could be difficult. Even if the packers are installed, the high grouting pressure may cause the packers to slide along the hole, as experienced in the Äspö tunnel. Therefore an alternative system with a T-valve as shown in Figure 6-2 is suggested, where the T-valve is coupled to the "blow-out preventer" that is installed during drilling operations. This system has the following advantages:

- Problems associated with the high water pressure during the installation of the traditional packers are eliminated, as well as packer sliding problems due to the high grouting pressure;
- Problems of large water inflow from the boreholes can be controlled more easily during the grouting operations;
- The T-valve can prevent clogging of the grouting equipment by opening the valve and pumping a small amount of grout onto the tunnel floor to get fresh material into the system.

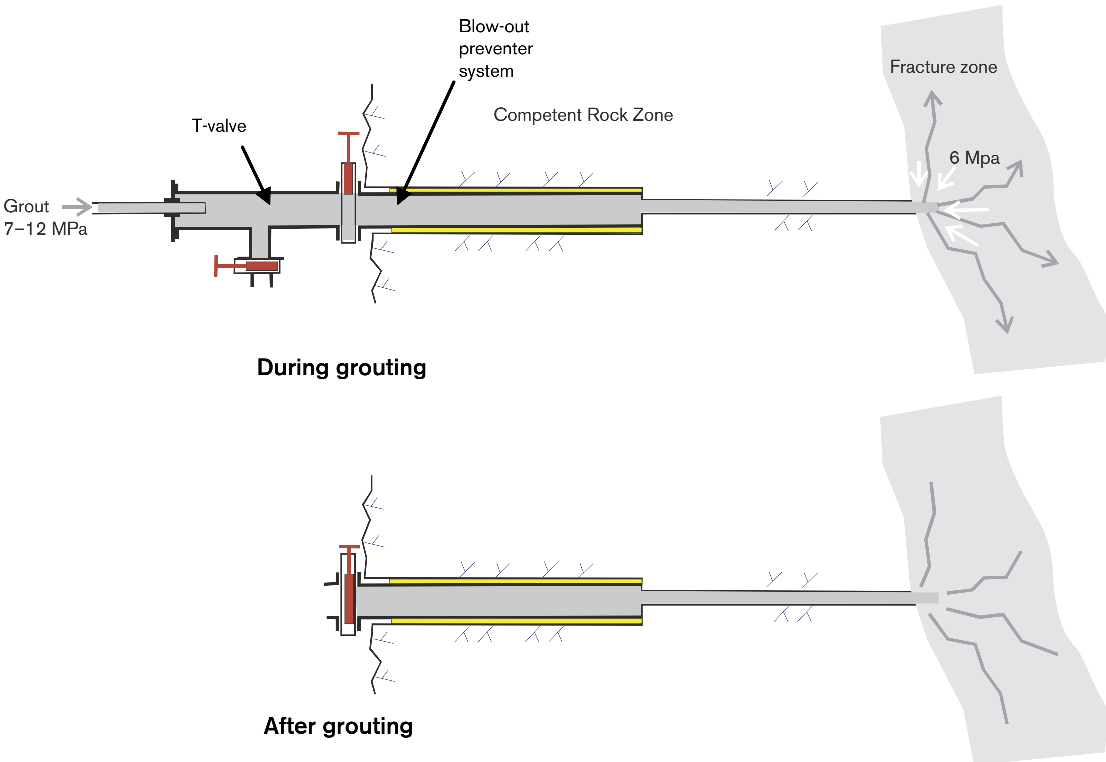


Figure 6-2. *A T-valve system for high pressure grouting.*

6.4 Execution of ground freezing

Though the freezing technique is well known and has been successfully used in tunnels worldwide, it is still a challenge to undertake freezing operations in water-bearing zones at great depths. The following construction related issues are of importance:

- Installation of freezing tubes under high water perssure;
- Limiting water movement due to tunnelling activities;
- Time required for the freezing operation;
- Concreting on frozen rock surfaces;
- Sealing arrangement for the permanent lining construction.

Due to the high water pressure in the fracture zone, the installation of freezing tubes must be done via long boreholes drilled from a safe distance. Casing of the boreholes must be employed to prevent borehole collapse and large water inflows.

Water movement in the ground will significantly deteriorate the freezing effects. Pre-grouting must be carried in front of the tunnel face as close to the fracture zone as possible to limit water movement towards the tunnel. A sketch is shown in Figure 6-3.

Casting of in-situ concrete on frozen ground must be undertaken carefully. The main considerations are /Harris, 1995/:

- The possible effect of the frozen ground on the ultimate quality, strength and water-tightness of the lining;
- The necessary precautions to be taken in placing concrete against frozen strata;

Research and construction works have shown that, providing certain principles are followed, a finished concrete lining which is homogenous, undamaged by the freezing action and durable throughout its design life can be achieved /Harris, 1995/. Good quality control of the concrete mix and rigorous supervision during concreting is essential. The heat balance at the point of contact is fundamental. The volume of concrete placed must generate sufficient heat of hydration to dominate the adjacent freezing action to allow the initial set

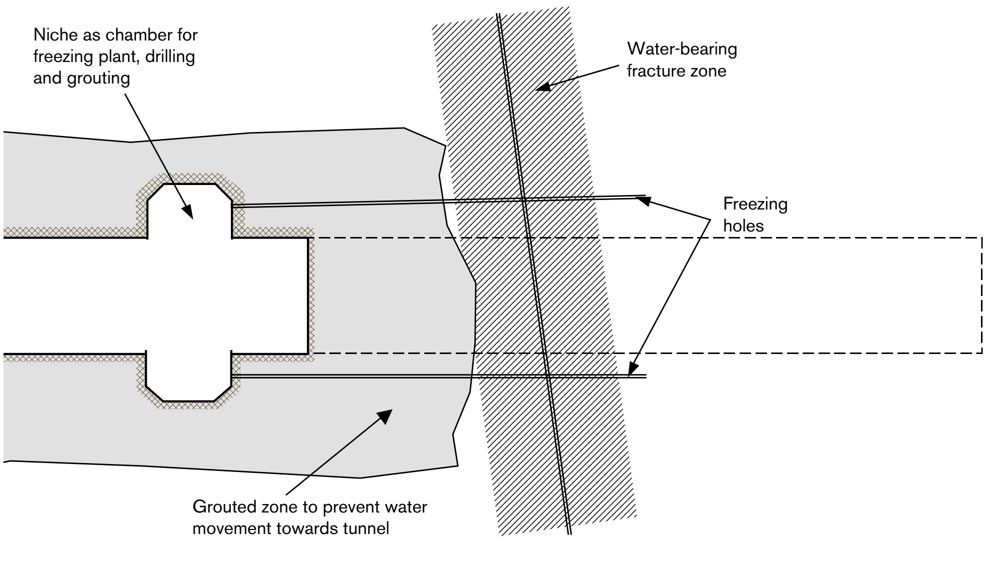


Figure 6-3. *Pre-grouting to limit water movement towards the tunnel.*

to proceed at a positive temperature. Another fundamental principle is to ensure the correct mix temperature of 19–20°C /Harris, 1995/. The sealing of the contact surface between the rock and the concrete lining must be carefully designed and constructed.

6.5 Rock supports of highly fractured rock

As described in Chapter 5, the core of the fracture zone is likely to be subjected to stability problems associated with large deformations at greater depths. However, the core has only a relatively short width (8 m for the case of this study) so that the stability problems can be managed with currently available methods for rock supports and tunnel excavations.

Rock bolting combined with shotcrete is considered as being sufficient to ensure the tunnel stability in the core of the fracture zone. Steel meshes embedded in shotcrete or spilling (forepole) could be necessary at greater depths. Flexible supports are preferable under rock conditions where large deformations are expected. A flexible support system with rock bolts and shotcrete, for example, has a greater capacity to accommodate large tunnel deformations, so that the stability is reached with an adequate load on the support. Otherwise, a stiff rock support will result in a high load on the support, sometimes even support failure.

Tunnels in highly fractured rock conditions often have limited stand-up times, which require rapid supports after blasting. Where necessary the shotcrete lining can be applied successively in layers in association with tunnel face advances (see Figure 6-4). This method was successfully used in the Uri-hydropower project in India, as reported by /Brantmark, 1997/. Figure 6-4 illustrates the method whereby the tunnel is supported with one layer of shotcrete at an early stage within the stand-up time. The thickness of the lining increases gradually in order to accommodate the tunnel deformations.

The study by /Chang, 1994/ has shown that the load on the shotcrete lining is strongly dependent on the tunnel excavation procedures. The advancement of the tunnel face is a loading process for the shotcrete. The study shows that if the speed of tunnel-advancement is high, there is a risk that the shotcrete could be over-loaded (see Figure 6-5). Such over-loading of shotcrete at early ages has negative effects on the subsequent strength development of the shotcrete. The hardening process of shotcrete must therefore be taken into consideration when determining the tunnel excavation procedures. A method for this purpose has been developed by /Chang, 1994/.

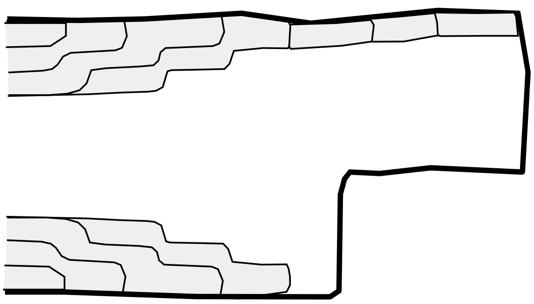


Figure 6-4. Shotcrete was applied in layers associated with the advances of the tunnel face /Brantmark, 1997/.

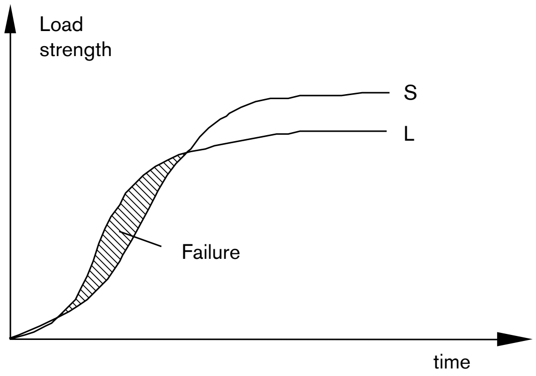


***Figure 6-5.** Early loading of shotcrete resulting in failure of the lining /Stille and Franzén, 1990/.*

It should be noted that it is a common practice in tunnel engineering that the shotcrete lining is not designed to sustain high water pressures. Due to water seepage through the grouted zone, water pressure can successively build up behind the shotcrete lining if the water is not drained away. Therefore shotcrete linings for a tunnel under high water pressure must be provided with a suitable drainage system and its function must be checked regularly.

6.6 Monitoring and back-analysis

Monitoring is an important part of modern tunnelling, with the aim partly to ensure that the requirements for stability and water inflow are being met, and partly to verify the engineering assumptions made in the tunnel design.

During tunnel excavation through a fracture zone, the monitoring program must at least include the following items:

- Measurement of tunnel deformations, i.e. extensometer and convergence measurements;
- Water pressure measurements in boreholes surrounding the tunnel;
- Examination of water leakage from the tunnel face;
- Water inflow measurements;
- Acceptable limits of behaviour regarding measured deformations and water inflows;
- Procedures for reporting the measured values;
- Procedures for contingency actions if the monitoring reveals behaviour outside acceptable limits.

Communication and cooperation between all the parties involved in a project plays an important role for a well functioning monitoring system.

Back-analyses are often carried out for verifying the design of the rock support system and providing information for adequate modification of the rock support system. The most common method of back-analysis is the use of numerical tools to match the measured deformations by varying the input of the numerical model. It has been proven that back analyses will provide valuable information about the behaviour of the tunnel.

Past studies have shown that the extension of the plastic zone around a tunnel is one of the important parameters for evaluation of rock mass behaviours around a tunnel. For instance, /Hoek, 2000/ proposed to use the extension of the plastic zones to estimate the risks for squeezing ground; while /Anagnostou and Kovári, 2003/ suggested to use it for the estimation of the deformation reduction factor for a fracture zone. As indicated in Chapter 5, the knowledge of the degree of plasticity inside a grouted/frozen zone is essential for the assessment of the stability of the grouted/frozen zone. The back-analysis as mentioned above could provide such information concerning plasticity around a tunnel. This approach is, however, quite time-consuming.

Another approach using strains to assess tunnel stabilities was first presented by /Sakurai, 1981, 1983/, where strains are obtained directly from the measured displacements. Comparison of the obtained strains with a so-called "critical strain" enables the determination of plasticity areas around the tunnel. This approach has the following advantages compared with the traditional approaches:

- The extension of the plastic zone can be determined quickly on site when new measurement data is available;
- No other parameters e.g. E, φ and cohesion of rock mass are needed in the calculations. The strains are derived directly from the measured deformations;
- This approach gives a higher level of certainty, because calculations are purely based on deformations, which are the only parameters that can be reliably measured during tunnelling.

However, the use of the so-called "critical strain" as a plasticity criterion is considered to be primitive. Testing data of rock samples /see e.g. Martin et al. 2001/ have shown that evident volume expansions are observed when the rock is subjected to the load at failure. Since the volume expansion can be expressed by the increment of the volumetric strains, the starting point for the increase of the volumetric strain, instead of the critical strain, may then be used as the plasticity criterion for the rock mass. This failure criterion can be named as "Rock Strain Strength". The principle of this approach using the rock strain strength is illustrated in Figure 6-6.

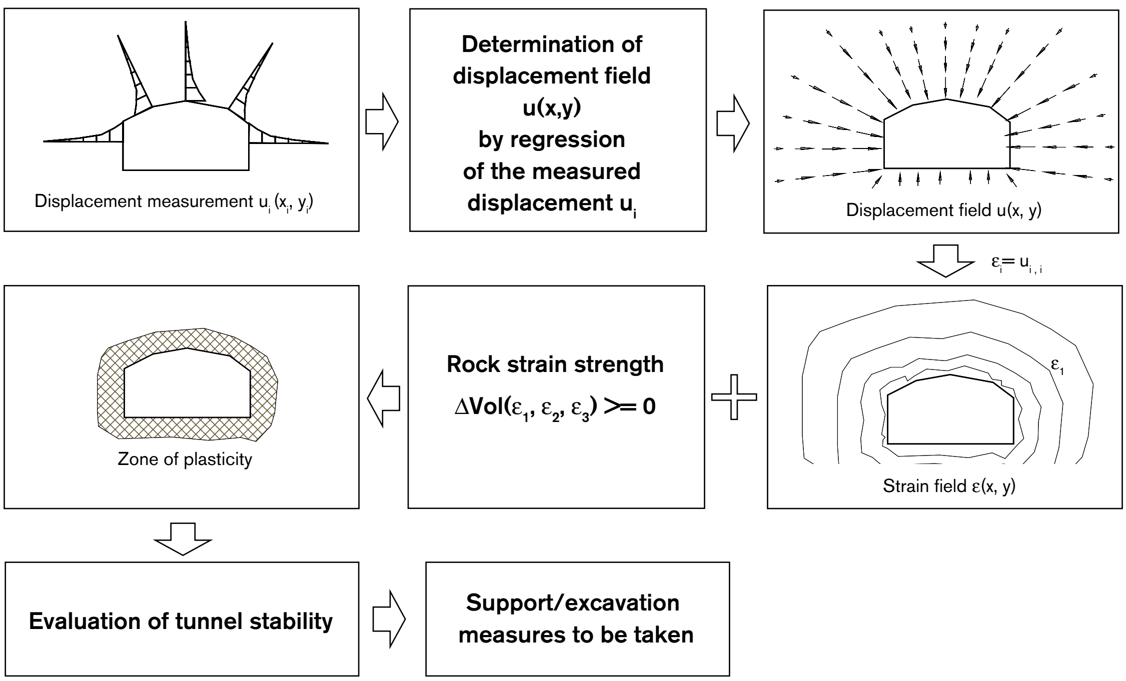


__Figure 6-6.__ An illustration of the strain approach for back-analysis and tunnel stability evaluation.

6.7 Summary

Probing is an essential part of modern tunnelling in order to minimize risks for unexpected rock conditions. Drilling operation in water-bearing zones requires special skills and it is advisable to contract specialists in the field. The T-valve system combined with a “blow-out-preventer” is preferable for grouting operations, so that problems associated with packer installation and sliding due to high grouting pressure can be eliminated. If ground freezing is employed, the placement of concrete lining on the frozen rock surface as well as the sealing arrangement between the rock and the lining must be carefully designed and constructed.

Flexible support with rock bolts and shotcrete is preferable for a tunnel with large deformations. Tunnel excavation rates and procedures must be adapted depending upon the hardening process of shotcrete. Drainage systems must be fitted to the shotcrete lining and regularly checked.

Communication, cooperation and trained personnel are important factors for well functioning probing and monitoring operations. Back-analysis of tunnel deformations is to be undertaken, preferably by means of the strain-based approach.

7 Hazard assessment

7.1 General

In the previous chapters, technical issues related to water inflows and tunnel stability were analysed. The analyses indicated that control of water inflow is the key issue for successful tunnelling through the water-bearing fracture zone. The aim of the hazard assessment conducted in this study is to provide an overall evaluation of the hazards associated with grouting and ground freezing.

7.2 Method of hazard assessment

During the process of this study, an attempt was made to employ fault trees and event trees to facilitate the hazard assessment. It has then been realized that a such detailed hazard assessment would be too complex to be included in this baseline study. It was found that an overview of technical hazards focusing on hazardous scenarios and their causes would be the appropriate level. For this objective, a simplified and scenario-based method is therefore employed for the hazard assessment (see Figure 7-1).

A hazardous scenario is defined in this study as an unwished incident that could lead to a catastrophic consequence for the tunnel excavation. A hazardous scenario could be a result of one or more hazard events that have possible causes in tunnelling operations. It is worth pointing out that natural causes such as highly fractured rock, high rock stresses and high water pressure are the preconditions for this study and they will henceforth not be quoted in the assessment.

Based on the analyses given in the previous chapters and discussions with tunnelling experts, the following two hazardous scenarios are of crucial importance to be considered for tunnelling through the fracture zone:

- Extremely large water inflows;
- Tunnel collapse.

In the following sections, the hazardous assessments considering the above hazardous scenarios are presented for outlining the hazardous events and their causes associated with grouting and ground freezing.

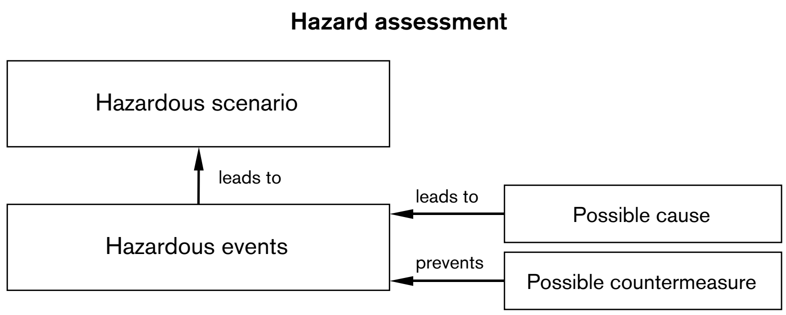


Figure 7-1. Method of hazard assessment used in this study.

7.3 Hazardous events associated with grouting

Possible types of tunnel collapse associated with grouting and their causes (hazardous events) are listed in Table 7-1 and illustrated in Figure 7-2, while Table 7-2 and Figure 7-3 show possible sources of water inflows and their causes.

Table 7-1. Possible types of tunnel collapse and hazardous events associated with grouting.

Hazardous scenario	Hazardous event
Collapse of the tunnel face before entering the grouted zone.	• The water-bearing zone is too close when grouting commences, so that large water inflow with high water pressure causes "flowing ground"; • Insufficient size of the grouted zone results in an insufficient sealing effect, so that large water inflow with high water pressure causes "flowing ground" when the face approaches the fracture zone; • Insufficient support of tunnel face; • Erosion of filling in fractures due to insufficient grouting reduces the strength of the rock mass.
Collapse of roof or walls during excavation through the grouted zone.	• Insufficient size of the grouted zone under high water pressure causes failure of the grouted zone; • Temporary or permanent rock support failures, due to e.g. difficulties in applying shotcrete on running-water surfaces.

Table 7-2. Possible sources of water inflows and hazardous events associated with grouting.

Hazardous scenario	Hazardous event
Large water inflow from borehole drilled for grouting.	• Defects in drilling operations, e.g. blow-out preventer is not functioning.
Large water inflow from tunnel face, roof, walls or floor.	• The tunnel face is too close to the water-bearing zone, so that water leaks through the fracture; • Unsatisfactory grouting quality gives insufficient sealing effects; • Insufficient size of grouted zone results in insufficient sealing effects.

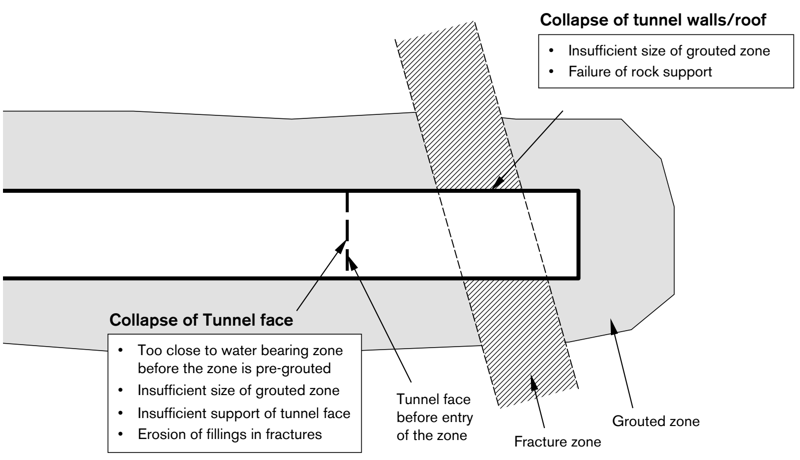


__Figure 7-2.__ Identified hazardous events associated with grouting which can lead to tunnel collapse.

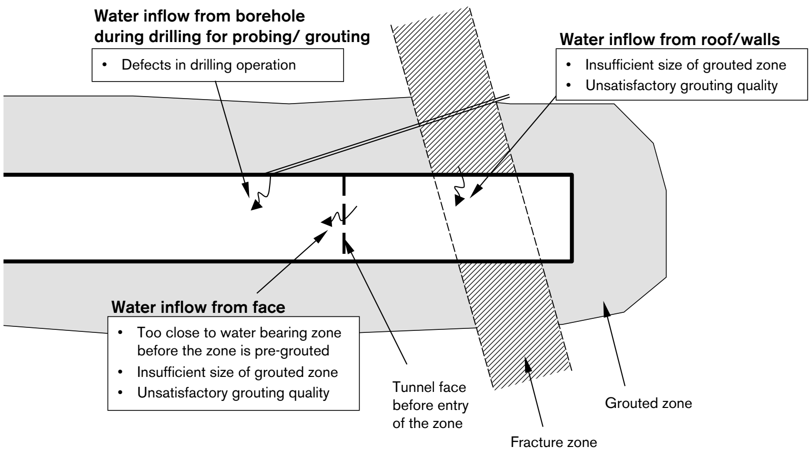


Figure 7-3. *Identified hazardous events associated with grouting which can lead to large water inflows.*

7.4 Hazardous events associated with ground freezing

Possible types of tunnel collapse associated with ground freezing and their causes (hazardous events) are listed in Table 7-3 and illustrated in Figure 7-4, while Table 7-4 and Figure 7-5 show the identified possible sources of water inflows and their causes.

Table 7-3. Possible types of tunnel collapse and hazardous events associated with ground freezing.

Hazardous scenario	Hazardous event
Collapse of the tunnel face before entering the frozen zone.	• The water-bearing zone is too close when freezing commences, so that large water inflow with high water pressure causes “flowing ground”; • Insufficient size of the frozen zone due to misjudgment of probing results, for example, gives insufficient structural support when the face approaches the frozen zone; the reasons for narrow frozen zone could be, for example, misjudgment of the width of the fracture zone; • Breakage of frozen zone due to shear deformation in overstressed rock mass; • Unsatisfactory freezing quality resulting in insufficient structural support when the face approaches the frozen zone;
Collapse of roof or walls during excavation through the frozen zone.	• Unsatisfactory freezing quality resulting in insufficient structural support; • Insufficient size of the frozen zone resulting in insufficient structural support; • Breakage of frozen zone due to shear deformation in overstressed rock mass; • Failure of temporary or permanent rock support

Table 7-4. Possible sources of water inflow and hazardous events associated with ground freezing.

Hazardous scenario	Hazardous event
Water inflow from borehole drilled for freezing tubes.	• Defects in drilling operations, e.g. blow-out preventer not functioning.
Water inflow from tunnel face before entering and passing the frozen zone.	• The face is too close to the water-bearing zone, so that water leaks through the facture; • Unsatisfactory freezing quality gives insufficient sealing effects; • The frozen zone does not cover the entire fracture zone, so that water leaks from the face when approaching the unfrozen section.
Water inflow from tunnel roof, walls or floor.	• Unsatisfactory freezing quality resulting in insufficient sealing effects; • Insufficient sealing of the permanent lining at rock contact.

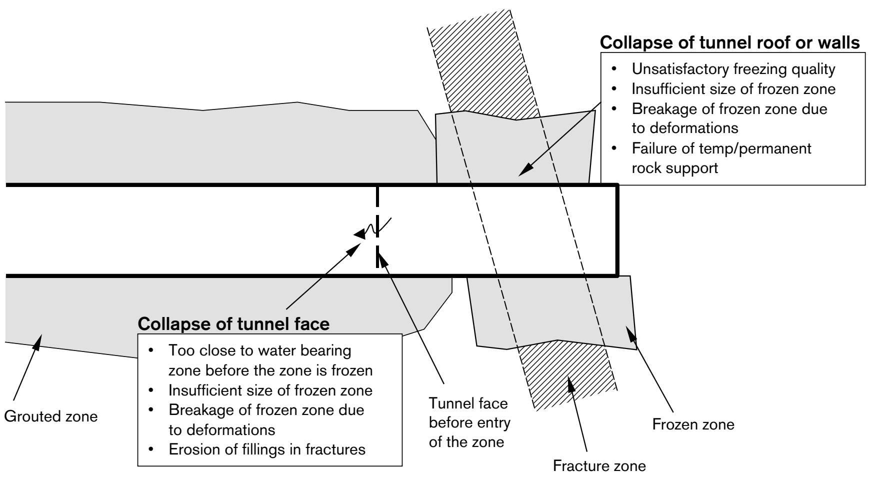


__Figure 7-4.__ Identified hazardous events associated with ground freezing which can lead to tunnel collapse.

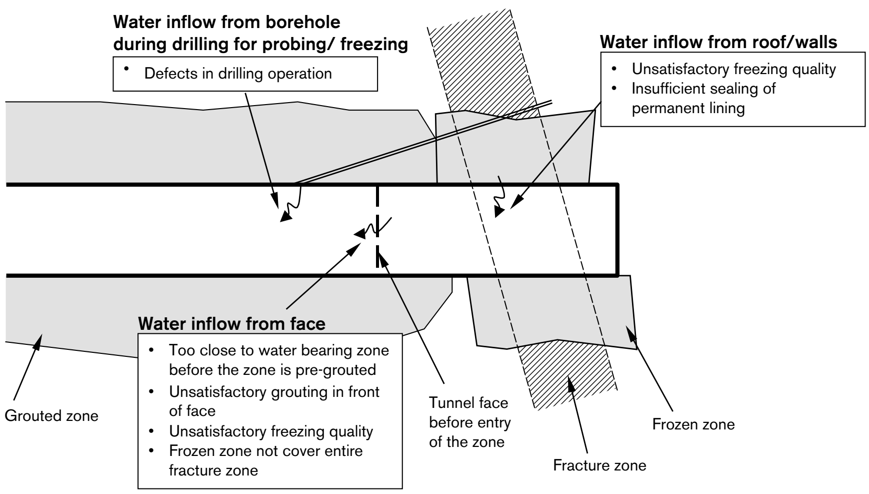


__Figure 7-5.__ Identified hazardous events associated with ground freezing which can lead to large water inflows.

7.5 Hazard assessment

After analysis and sorting of the results given in the previous sections, the following major hazardous events are recognised for grouting respectively for ground freezing:

Hazardous events associated with grouting:

1. The water-bearing zone is too close when grouting commences;
2. Insufficient size of the grouted zone;
3. Failure of temporary or permanent rock support;
4. Water inflow from boreholes;
5. Unsatisfactory grouting quality.

Hazardous events associated with ground freezing:

1. The water-bearing zone is too close when freezing commences;
2. The frozen zone does not cover the entire fracture zone;
3. Temporary or permanent rock support fails;
4. Water inflow from boreholes;
5. Unsatisfactory freezing quality;
6. Insufficient sealing of permanent lining.

The assessment of these hazardous events and their major causes, possible countermeasures, impacts on tunnel stability and water inflows are summarized in Table 7-5 for grouting and Table 7-6 for ground freezing. These tables can be used for e.g. development of an "operational tunnelling protocol" in the coming stages of SKB's design process.

Table 7-5. Overall hazard assessment associated with grouting.

Hazardous event	Possible cause	Possible countermeasures	Impact with regard to large water inflow	Impact with regard to tunnel collapse
1. The water-bearing zone is too close when grouting commences.	• An unexpected zone is encountered.	• Proper probing and well established routines for interpretation and reporting.	• Water flows from the face causes problems for grouting and working conditions deteriorate.	• Face instability might cause flooding of tunnel (flowing ground).
2. Insufficient size of grouted zone.	• Limited knowledge of grout spreading in rock or poor performance.	• Careful design and verification at site.	• Large water inflows into tunnel.	• Instability of tunnel walls and faces due to high water pressure.
3. Temporary or permanent rock support fails.	• Poor design or performance.	• Monitoring and re-evaluation of design if required.	• Large water inflows if tunnel collapses.	• Collapse of tunnel.
4. Water inflow from boreholes.	• Faults in equipment including blow-out-preventer.	• "Blow-out preventer" and skilled personnel.	• Deterioration of working conditions for personnel.	
5. Unsatisfactory grouting quality.	• Limited knowledge of grouting in fractured rock at great depths or poor performance.	• Careful design and verification at site.	• Large water inflows into tunnel.	• Erosion of rock mass might cause tunnel instability.

Table 7-6. Overall hazard assessment associated with ground freezing.

Hazardous event	Possible cause	Possible countermeasures	Impact with regard to large water inflow	Impact with regard to tunnel collapse
1. The water-bearing zone is too close when freezing commences.	• Unexpected zone is encountered.	• Proper probing and well established routines for interpretation and reporting.	• Water flows from the face causes problems for freezing operations and working conditions deteriorate.	• Face instability might cause flooding of tunnel (flowing ground).
2. Frozen zone not covering the entire fracture zone.	• Misjudgment of the width of fracture zone; unexpected water movement in rock mass.	• Careful probing and interpreting probing results.	• Water inflow from the face when approaching the zone.	• Face instability (flowing ground) when approaching the zone.
3. Temporary or permanent rock support fails.	• Poor design or performance.	• Monitoring and re-evaluation of design if required.	• Large water inflows if tunnel collapses.	• Collapse of tunnel.
4. Water inflow from boreholes.	• Faults in equipment including blow-out-preventer.	• "Blow-out preventer" and skilled personnel.	• Deterioration of working conditions for grouting.	
5. Unsatisfactory freezing quality.	• Unexpected water movement in rock mass; faults in freezing operation.	• Proper probing and skilled personnel, monitoring of temperature.	• Successive increase of water inflow might result in tunnel collapse.	• Tunnel collapse due to high water pressure on the frozen zone.
6. Insufficient sealing of permanent lining.	• Faults in design or performance.	• Quality control in design and performance; skilled personnel.	• Large water inflows after tunnel construction leads to high costs for tunnel operation.	• Long term erosion of fillings in fractures might cause tunnel instability.

8 Suggested tunnelling approach

8.1 General

Based on the technical analyses and hazard assessment in the previous chapters, this section will present the proposed approaches for the tunnel excavation, with the aim of minimizing the risks of passage through the fracture zone.

As indicated in the previous chapters, the passage of a tunnel through a water-bearing fracture zone with the characteristics of NE-1 at a depth of 600 metres presents numerous technical challenges. Precautions must therefore be taken in the decision making process.

Experience from the Äspö tunnel where NE-1 was encountered indicates that careful probing before entering the facture zone provided valuable information about the characteristics of the zone. This played an important role in the planning, design and excavation through the zone. The experiences from the Äspö tunnel as well as other case histories are incorporated in the proposed excavation approach.

8.2 Description of tunnelling approach

A general description of the tunnelling approach for all depths is shown in Figure 8-1 and is briefly described as follows. The overview flowcharts are presented in Figure 8-2 for grouting and Figure 8-3 for freezing.

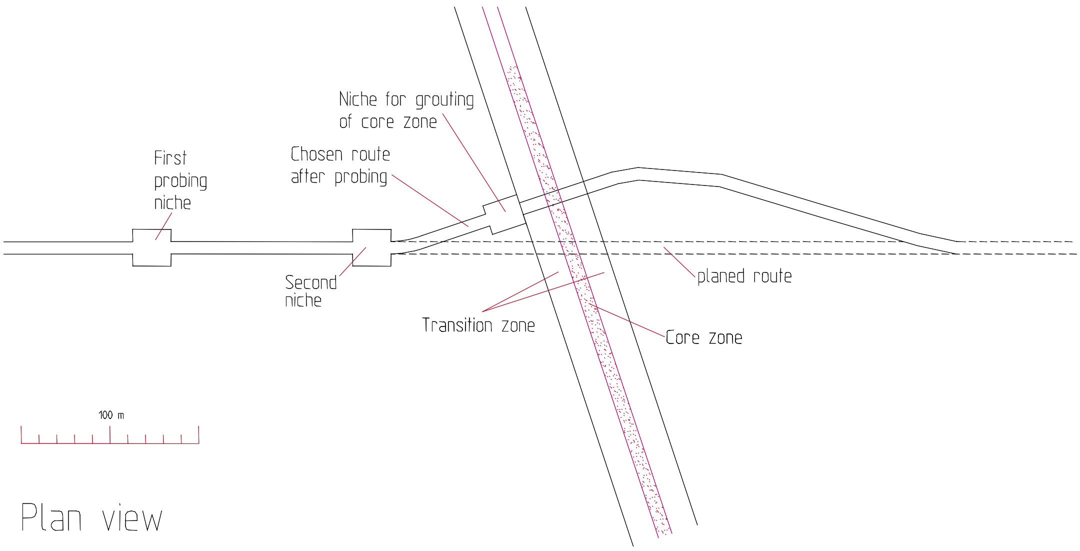


Figure 8-1. *Suggested tunnelling approach.*

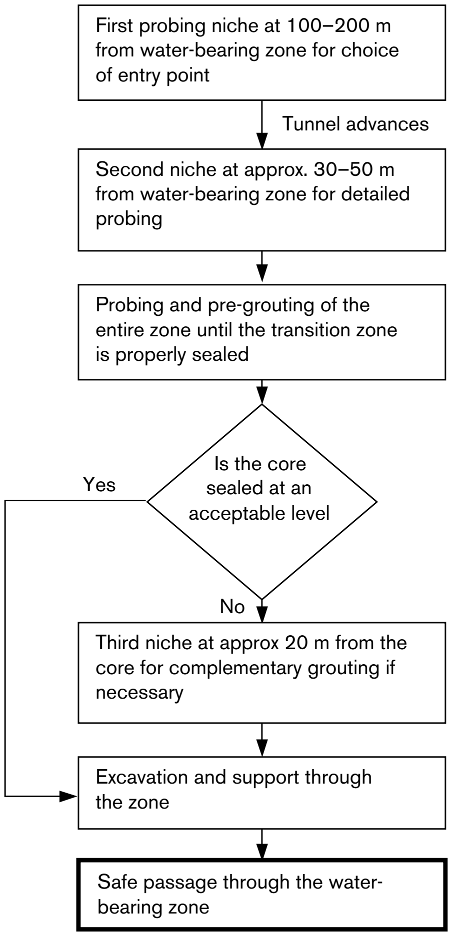


Figure 8-2. *Overview workflow for tunnelling through water-bearing fracture zones.*

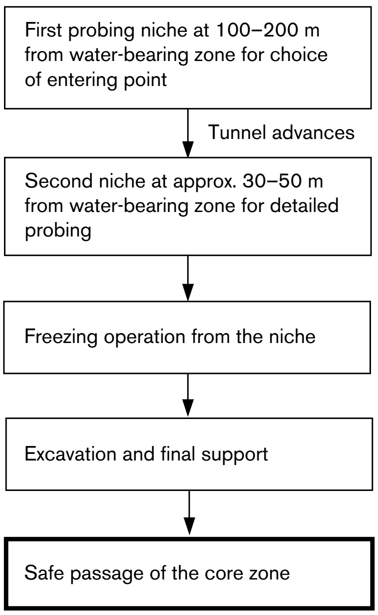


Figure 8-3. *Overview workflow for tunnelling through the core zone using ground freezing.*

1. A niche is excavated at approximately 100–200 metres from the water-bearing zone. Probe drilling is carried out to define the location and characteristics of the zone. The drilling equipment must be capable of sustaining the prevailing water pressure and boreholes must be equipped with "blow-out preventers" to control the water-inflow from the boreholes;

2. Based on the results of probing, the location for the point of entry for passage through the fracture zone can be preliminarily determined. Since the characteristics of a fracture zone may vary significantly over a relatively short distance, it is worth evaluating several alternatives for the entry point. For instance, once the fracture zone is allocated in detail by probing, it could be advantageous to relocate the route of the access tunnel so that it enters the zone perpendicularly. After passing the fracture zone, the direction of the access tunnel can be readjusted to its original route.

3. The access tunnel is excavated to a safe distance from the zone (about 30–50 metres) and a second niche is made. Detailed probing is then performed and details for water inflow control can hence be determined. Pre-grouting is conducted from the niche and penetrates the entire water-bearing zone.

   For tunnel excavation at a depth of 600 metres, a decision may be necessary on whether the core is to be frozen or grouted. If the decision is to freeze the core, pre-grouting is conducted from the niche to seal the transition zone and freezing operations are prepared. If grouting is chosen, pre-grouting is conducted from the niche to penetrate the entire water-bearing zone. It is worth pointing out that the decision of freezing should be made as early as possible and the design related to freezing must be conducted before the excavation works reach the fracture zone.

4. If there is a need for the core zone to be complementarily pre-grouted, a third niche could be necessary at a closer distance to the core zone.

5. When the requirements of water inflow have been satisfied by pre-grouting or ground freezing, the tunnel can be driven through the water-bearing zone with necessary supports or concrete lining.

Advantages with the proposed excavation strategy are as follows:

- Probing from the first niche will give valuable information concerning the water-bearing zone and a favourable entry point can then be chosen preliminarily. The risk of serious problems associated with large water inflows and high water pressures can be assessed at an early stage; this will allow decisions to be made on necessary groundwater control measures and equipment;
- The probing process is an information gathering process, such that appropriate decisions can be made based on new information; this ensures a high degree of flexibility so that control measures can be matched to actual site conditions.

Proposals for rock supports for the various depths are given in Table 8-1, based on the stability analyses relating to large deformations given in Chapter 5 and the suggested support types of /Hoek, 2000/ (see Table 5-5).

At a depth of 600 metres, another alternative could be considered if the access tunnel is excavated as a downward spiral; namely that pre-grouting or freezing of the water-bearing zone can be performed in an adit at higher level of the spiral (Figure 8-4).

Table 8-1. Rock supports for passage of the water-bearing fracture zone.

	Transition zone	Core of the zone
200 m	Rock bolts and steel-fibre-reinforced shotcrete	Rock bolts and steel-fibre-reinforced shotcrete with wire mesh embedded
400 m	Rock bolts and steel-fibre-reinforced shotcrete	Rock bolts and steel-fibre-reinforced shotcrete with wire mesh embedded. The face is supported when necessary.
600 m	Rock bolts and steel-fibre-reinforced shotcrete. Wire mesh is embedded in shotcrete when necessary.	Rock bolts and steel-fibre-reinforced shotcrete with wire mesh or lattice girders embedded. The face is supported when necessary. Concrete lining if ground freezing is used.

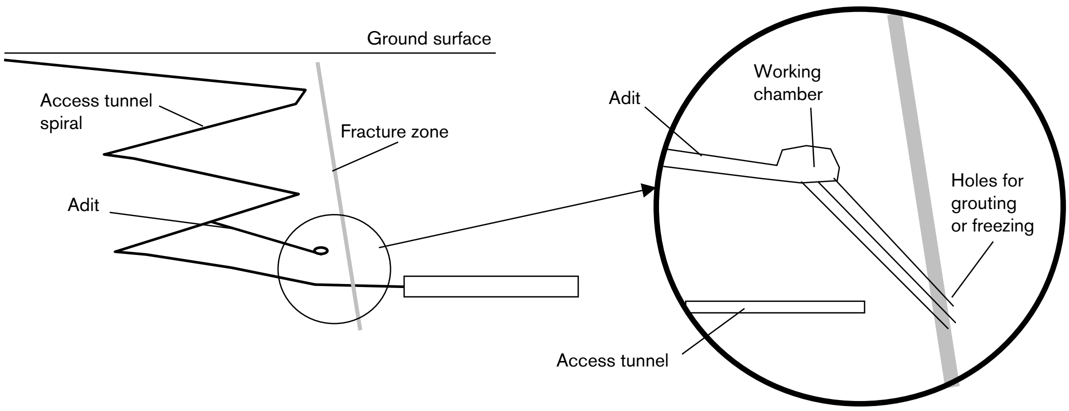


Figure 8-4. *Another alternative for water inflow control at great depths: Pre-grouting or freezing of the water-bearing zone from an adit at higher level.*

One of the advantages of this alternative is that the operations for pre-grouting /freezing are carried in the adit, while the access tunnel is advancing. The water-bearing zone will have been treated prior to the access tunnel reaching the zone. In this way the whole construction time can be shortened. Another advantage is that the water pressure in the boreholes drilled from the working chamber will be lower compared with the boreholes drilled from the tunnel, so that difficulties associated with drilling may be reduced.

9 Conclusions

9.1 General

This chapter presents conclusions and recommendations based on this baseline study regarding the important technical issues associated with tunnelling through a water-bearing fracture zone at great depths. The conclusions are drawn from the results of the analyses and discussions as presented in the previous chapters, in the context of the control of water inflows, tunnel stability and tunnel construction. The study presented in this report is essentially undertaken on the basis of the descriptive model originated from the characteristics of the deformation zone NE-1. The conclusions given in this report could thus be applicable if an actual fracture zone has similar characteristics as that given in the descriptive model. Nevertheless, the methods for the analyses presented in this report are general and can be applied for problems where the preconditions for using the methods are valid. It is worth noting that there are other methods and tools, for instance numerical computer programs, that can be used for detailed studies.

At this stage, the content of the report, the conclusions and recommendations should not be interpreted as necessarily being SKB's view of approaches to the technical solutions for tunnelling through such a fracture zone.

9.2 Conclusions

Analysis and discussions regarding the technical issues of water flow control, tunnel stability and construction are presented in Chapter 4, 5 and 6. The hazard assessment and proposal for tunnelling approaches are given in Chapter 7 and 8. Conclusions drawn from these results as follows:

1. Control of water inflow is the key issue for executing a safe tunnelling project through the water-bearing fracture zone with the characteristics as described in the descriptive model. The experience of the passage of NE-1 showed that the reported post-grouting water inflows for the sections where NE-1 was encountered were approximately 50 l/min/10 m. The analysis indicates that the achieved hydraulic conductivity of the grouted rock in NE-1 is likely to be in the range of 5 to 8×10^{-8} m/s. In order to reduce the water inflow to 50 l/min/10 m for the scenarios used in this study, the required hydraulic conductivity after grouting is likely to be in the range of 2 to 8×10^{-8} m/s, depending on the tunnel depth. These values of hydraulic conductivity can be achieved using current grouting technology.
2. A possible alternative to water inflow control is ground freezing. The choice of ground freezing must, however, be carefully analysed and must take into account risks, costs and time schedules. In the detailed design of ground freezing, the structural design must ensure tunnel stability during the tunnel excavation, while thermal design is to determine the freezing capacity required. Groundwater flow and salt content in the groundwater are the two major hindrances with regard to the freezing process. Monitoring during the entire freezing process is essential in order to ensure that the frozen zone performs according to the design.

3. At great depths, a high water pressure would probably act on the outer boundary of a grouted zone. The thickness of the grouted zone must therefore be sufficiently large in order to sustain the water pressure. Moreover, the stresses induced by the water pressure gradient inside of the grouted zone must be carefully evaluated. The latter issue has yet to be studied in depth and further research in this field is recommended.

   If ground freezing is to be used, the thickness of the impermeable frozen zone is of importance, as it must be sufficient to bear the water pressure equivalent to the initial water head. Design of the permanent lining to ensure long-term stability must take into consideration both the ground pressure and the water pressure at the depth in question. Preliminary estimates indicate that the required thickness of the permanent concrete lining should be approximately 2 metres at a depth of 600 metres, assuming the lining is impermeable.

4. The deformation analysis of the tunnel subjected to the overstressed rock indicates that large deformations are unlikely to occur in the transition zone, even at a depth of 600 metres. The reduction in rock mass quality in the core zone, however, is likely to result in large deformations at great depths. The estimated mean values of deformation for an unsupported tunnel in the core zone are 60 mm and 130 mm at depths of 400 and 600 metres respectively. The stability problems associated with such deformations can essentially be dealt with using conventional support methods in form of rockbolts and fibre-reinforced shotcrete, however, the introduction of steel sets and the use of forepole umbrellas may be necessary.

5. Tunnel face stability is of crucial importance for the safety of tunnel excavations under high water pressure. For instance, face instability in form of flowing ground would result in catastrophic consequences, e.g. flooding of the whole tunnel, which may jeopardise the whole tunnel project. Nevertheless, such risks can be decreased considerably by careful probing ahead of the tunnel face. Another effective measure for preventing the face from collapse due to high water pressures is close observation of the face. Large water leaking from fractures at the face is an early indication of the presence of water behind the face. The importance of examination of the tunnel face for water seeping from fractures should therefore be emphasized during tunnelling operations.

6. Based on the analyses given in the previous chapters and discussions with tunnelling experts, two hazardous scenarios, namely 1) extremely large water inflows and 2) tunnel collapse, must be carefully evaluated for tunnelling through the fracture zone. The occurrence of one of the scenarios could lead to serious consequences, e.g. flooding of the tunnel. Major hazardous events that could lead to such dangerous scenarios are identified and the results could be used for the tunnel design and construction.

7. Drilling and grouting operations under high water pressure can be troublesome. The use of a "blow-out preventer" and T-valves could prevent problems associated with high water pressure, for instance the installation of packers. Choice of grouts with high early strength would be preferable to avoid the grouts being "washed-out" by the high water pressure. More detailed investigation on grouting in fractured rock under water pressure, e.g. equipment, types of grout and grouting effects, are recommended.

It is worth pointing out that the conclusions given above regarding water inflows are based on the assumption that the hydrogeological properties included in the descriptive model are the same at all depths. This assumption is made due to lack of depth specific information. Nevertheless, observations in Swedish deep ore-mines indicate that water inflows decrease at greater depths. One possible explanation could be that fractures are closed by higher rock stresses. The solutions proposed in this study can therefore be optimized when more detailed information is available for the fracture zones at great depths.

9.3 Recommendations for further investigations

This baseline study will provide a base for further studies and investigations for technical solutions of tunnelling through a water-bearing fracture zone at great depths. The following recommendations are presented showing some of important issues that require further investigation:

1. Further investigation of the characteristics of water-bearing fracture zones is needed for tunnel design purposes. Some of the important parameters are given as below:
   - Rock mechanics properties within fracture zones, e.g. rock mass strength, modulus of deformation, fracture properties, time-dependent behaviour of fractured rock mass, rock stresses within facture zones etc;
   - Hydrogeological parameters, e.g. hydraulic conductivity, water storativity, hydraulic contacts between fractures zones, hydraulic contacts with the sea; ground water flows, salt contents in the ground water etc;
   - Depth dependency of the above parameters.
2. The importance of the stability of the tunnel face should be highlighted. A safe distance to the fracture zone with respect of a high water pressure in front of the face must be further studied. Procedures for observations at the tunnel face must be established before the tunnel excavation commences, in order to prevent serious consequences due to tunnel face failure.
3. The difficult task of grouting in highly fractured rock under high water pressure should be recognized. The engineering practise of grouting, in the context of the effects that can be achieved, suitable equipment, choice of grout types, grouting pressure etc, should be investigated further.
4. Stability of grouted zones under high water pressure, considering the high water pressure on its outer boundary as well as the stresses caused by the hydraulic gradient inside of the grouted zone, should be well understood. This issue is a coupled problem between rock mechanics and hydrogeology. Relationships between stresses in the rock and water seepage around a tunnel are of considerable importance for the stability of the grouted zone. Studies by means of numerical analysis as well as model testing are recommended.
5. Difficulties with determination of rock mass properties in deformation zones for engineering purposes must be realized. It has been shown in this study that the values of the rock mass strengths are strongly dependent on the method(s) used. For the purpose of engineering design of an underground project, a methodology for determination of rock mass properties must be established at an early stage and must be consistently followed throughout the entire project, because such a methodology has significant impact on the design, construction as well as contractual issues. Methodology for determination of rock mass properties have been discussed by /Andersson et al. 2002/ and /Hudson, 2002/. It is stated by /Hudson, 2002/ that data should, if possible, be collected in the vicinity of the planned location of the excavations.

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Appendix 1

A1 Review of SKB's reports regarding NE-1

A1.1 Scope

A significant amount of data has been assembled both prior to, and after, the passage of deformation zone NE-1 during the construction of the Äspö HRL. The reports/data studied include pre and post technical notes, progress reports, technical reports and up-to-date site descriptive models that use 3D modelling methods. This appendix is essentially a summary of the information contained within these reports and has been used as the basis for the descriptive model presented in Chapter 2. The reviewed data has been divided into 5 discrete sections; geology, rock mass properties, *in situ* rock stress, hydrogeology and experience gained from Äspö HRL.

Italic text has been used for text that has been taken directly from the original report without modification.

SKB's geological terminology has been standardized during the ongoing site investigations and future reference will incorporate the new nomenclature. A reference for the old and new geological terminology is given in Table A1-1.

Where a previously published document is quoted directly, the original geological terminology has been retained to maintain the report's authenticity and avoid misinterpretation of previous information.

Table A1-1. Reference between old and new geological terminology.

Terminology used in previous SKB reports	New SKB Standard terminology
Diorite	Quartz monzodiorite (Äspödiorite)
Aplite	Fine-grained granite (Aplite)
Småland granite	Ävrö granite (Småland-Ävrögranite)
Greenstone	Fine-grained diorite-gabbro (Greenstone)

A1.2 Geological description of NE-1

A1.2.1 General

The geological information presented in this section has been taken from the following reports:

- TN-25-92-12N "Passage of waterbearing fracture zones. Passage of NE-1. Construction work including rock reinforcement, grouting and tunnel advance." /Hamberger and Didriksson, 1992/.
- TN-25-92-19N "Passage of waterbearing fracture zones. Passage of NE-1. Tunnel stability and recommendations for rock support at NE-1." /Stille and Olsson, 1992/.
- TN-25-92-15N "Passage of waterbearing fracture zones. Passage of NE-1. Evaluation of grouting activities at zone NE-1." /Stille et al. 1992/.
- PR-25-92-18 "Passage through water-bearing fracture zones. Evaluation of investigations in fracture zones NE-1, EW-7 and NE-3." /Rhen and Stanfors, 1993/.

- TR-97-06 "ÄSPÖ HRL. Geoscientific evaluation 1997/5. Models based on site characterization 1986–1995." /Rhen et al. 1997a/.
- PR-HRL-96-19 "Overview of documentation of tunnel, niches and cored boreholes." /Markström and Erlström, 1996/.

A1.2.2 Review of geological descriptions

Deformation zone NE-1 was initially located through surface geophysical methods and borehole radar. As the site investigation progressed, the zone was intersected by a number of boreholes (KAS 09, 11, 14 and KBH 02) /Stille and Olsson, 1992/.

The zone was later encountered during the construction of the access tunnel to the Äspö Hard Rock Laboratory. Before NE-1 was entered, the zone was carefully characterized and a description and classification of the rock mass within NE-1 was given in /Stille and Olsson, 1992/. A summary of this is provided in Table A1-2.

Table A1-2. Predicted rock mass classes for deformation zone NE-1 (for an explanation of rock mass classes see section A1.3.3) /Stille and Olsson, 1992/.

	Diorite	Aplite	Total
Class B Good rock	40%	5%	45%
Class C Fair rock	15%	10%	25%
Class D Poor rock	10%	20%	30%
Total	65%	35%	100%

Based on information from the site investigation, the deformation zone was interpreted to trend NE/SW and dip 50–60° NW. The borehole logs documented the zone as a series of highly fractured "sub zones" with possibly one highly fractured centre zone. The rock type was thought to vary between diorite and aplite.

The highest fracture frequencies were anticipated to occur in the aplite, in which RQD values as low as 0 were reported from borehole cores. The joint surfaces in the aplite were assumed to have reduced friction, however, fracture fillings such as chlorite, which reduce friction dramatically, only occurred sparsely.

The diorite was interpreted to be generally less fractured than the aplite, however, it was found to be partly mineralogically altered. Fracture fillings of chlorite and calcite were common in the diorite.

Based on the core logs from KAS 09 and KBH 02, the zone was assumed to be 50–70 m wide and have the distribution of rock types as shown in Table A1-2.

The predicted geological model suggested that NE-1 would be built up of a number of smaller sub-zones of fractured rock, between which, would lie good quality rock.

Following the passage of the zone, a description of the geological character of NE-1 was given and can be found in /Stille et al. 1992/. The description is as follows:

NE-1 proved to be highly waterbearing and is assumed to trend 060 and dip 70° to the north.

The most intense part of the zone, which intersects the tunnel at 1/300 m, is approximately 5 m wide, highly fractured or crushed and partly clay altered (smectite according to the analysis). This intense part of the zone with open, centimetre-wide fractures and cavities is surrounded by 10–15 m wide sections of more or less fractured rock. The tunnel intersects the zone along a length of approximately 30 m.

The main rock type in the zone is Småland granite with minor inclusions of greenstone and mylonite while the most intense part of the zone is located in a 10 m wide section of fine-grained granite.

A set of highly waterbearing structures – gently dipping towards the north – contribute to the complex character of the zone NE-1.

A similar and slightly more expansive description of the geology of deformation zone NE-1 is included in /Rhén and Stanfors, 1993/, which is as follows:

NE-1 proved to be highly waterbearing and is assumed to trend 060 and dip 70° to the north. The most intensive part of the zone, which intersects the tunnel at 1/300 m, is approximately 5 m wide, highly fractured or crushed in an approximately 1 m wide section partly clay-altered. The gouge material includes fragments of all sizes from centimetre scale down to < 0.125 mm. The fragments are sharp and angular.

More or less tectonized granite and mylonite occur. Older fracture formations are also found as fragments, indicating that the gouge formation is a reactivation of an earlier zone, which was developed under ductile-semiductile conditions.

Some fragments are penetratively oxidized, probably before the fragmentation took place and a post-fragmentation precipitation of pyrite on the grain surfaces is visible indicating that reducing conditions prevail.

The clay mineral present in the gouge is mainly mixed-layer illite/smectite. Very fine-grained quartz and feldspar also occur in the clay fractions.

The intensive part of the zone with open, centimetre-wide fractures and cavities is surrounded by 10–15 m wide sections of more or less fractured rock. The tunnel intersects the zone along a length of approximately 30 m. The main rock type is Småland granite/Äspö diorite with minor inclusions of greenstone and mylonite whilst the most intensive part of the zone is located in a 10 m wide section of fine-grained granite. A set of highly water-bearing structures – gently dipping towards the north – contribute to the complex character of the zone.

Included in /Rhén and Stanfors, 1993/ is an analysis of the structural fracture array taken in relatively close proximity to NE-1 (tunnel stretch 1,035–1,260 m). Several different fracture sets were identified and stereographic projections for these fracture sets are presented in /Rhén and Stanfors, 1993/. As the information from this fracture set analysis does not relate to deformation zone NE-1 itself, it has not been included in the study.

A later report, /Rhén et al. 1997a/ provides a summary of the geology of deformation zone NE-1 zone. The description is presented below.

On the site scale the major fracture zone NE-1 is interpreted to trend 50–60° NE and dip 70–75° NW. The zone is estimated to be 60 m wide, consisting of 3 branches. All three branches are connected to a rather complex rock mass with Äspö diorite, fine-grained granite and greenstone. The major part of the zone may be approximated by two planar water-bearing branches.

The two southernmost branches, trending NE and dipping N, can be described as highly fractured and more or less waterbearing. The northern branch, which is approximately 28 m wide in the tunnel, is the most intense part of NE-1 and highly water-bearing. An approximately 8 m wide part of this branch trending N 50° and dipping 75° N, with open, centimetre wide fractures and cavities and partly clay-altered rock, is surrounded by 10–15 m wide sections of more or less fractured rock. The central 1 metre wide section is completely clay altered.

The fractures within NE-1 did not in any respect differ from the average rock mass at the HRL except from a slight increase in epidotic/quartzitic coatings and minor (< 1 cm wide) mylonitic shear zones. The margins of various strands of NE-1, some of which displayed faults with gouge, are characterized by steep gradients in fracture frequency. The geometry of NE-1 constituent fractures could not be investigated in detail due to safety considerations during construction and extensive reinforcement. However, the results indicate that the fracture array consists of three sets; one is steep and strikes WNW, one is sub-horizontal and a third dips moderately towards NNW. The latter contains long faults with gouge and has been used to determine the local orientation of NE-1.

/Rhén et al. 1997a/ also include a 3D projection of the deformation zone, which was originally presented by /Munier, 1995/ and is shown in Figure A1-1.

Mapping of the geology was conducted during the driving of the access tunnel. The logs of the entire tunnel are presented in /Markström and Erlström, 1996/. The geological log for the stretch of tunnel that includes NE-1 is presented in Figure A1-2.

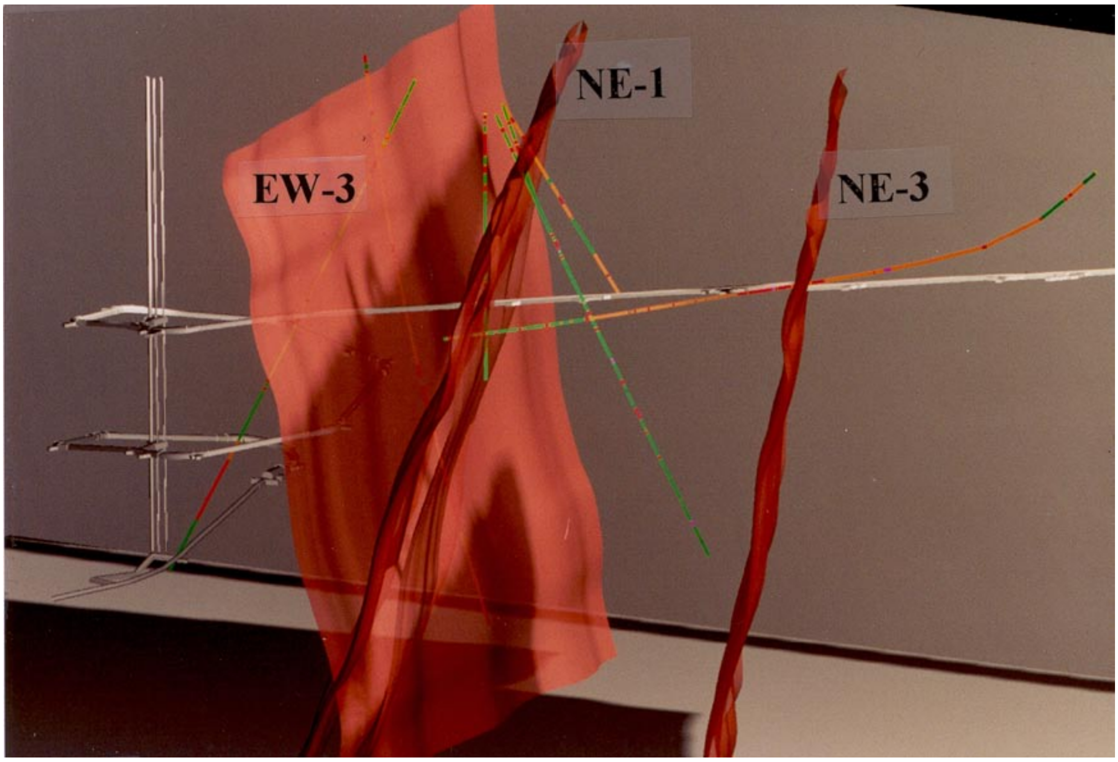

Figure A1-1. *3D graphics showing the geometries of NE-3, NE-1 and EW-3 /Rhén et al. 1997a/.*

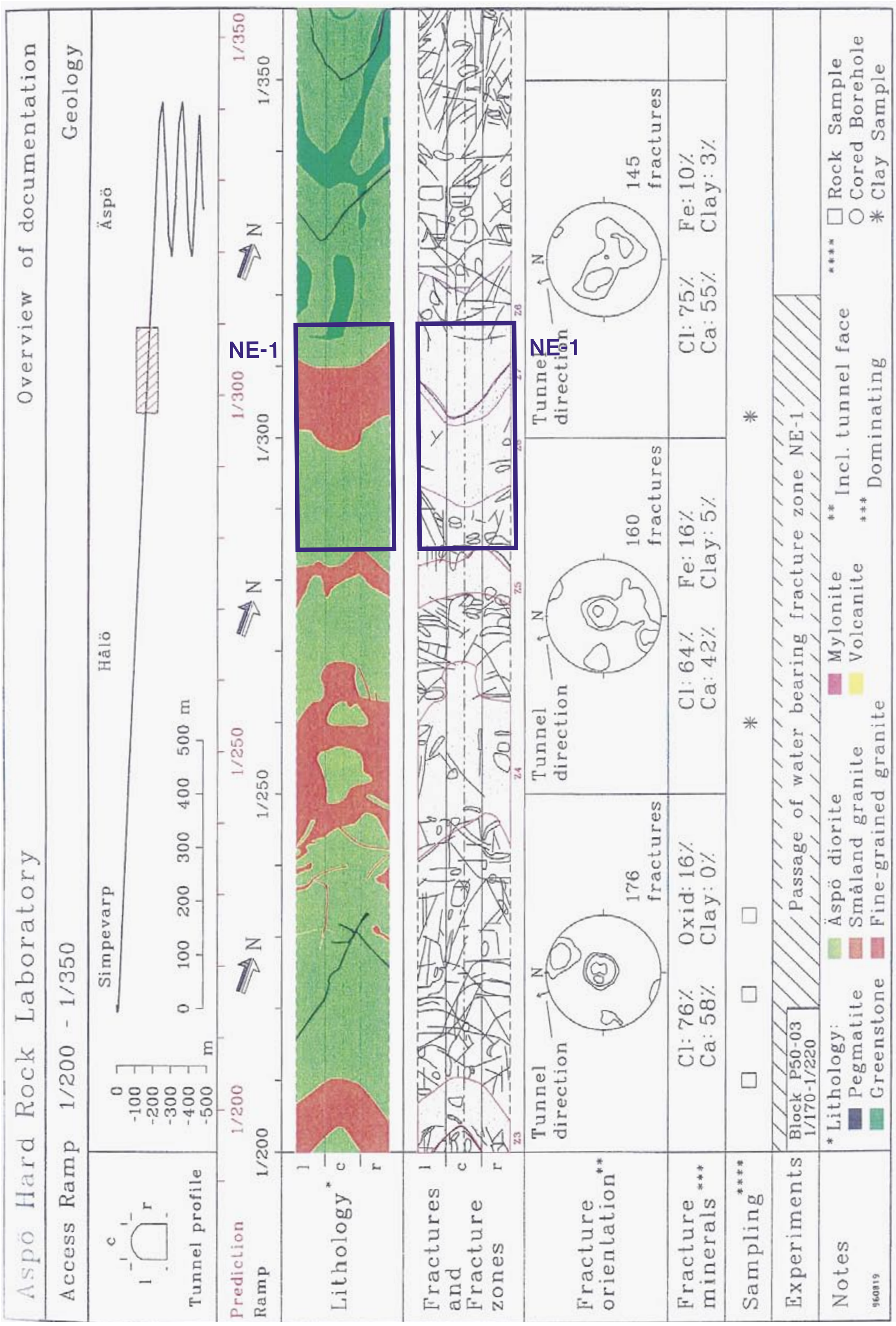

Figure A1-2. *Geological mapping of the tunnel. The blue area marked NE-1 shows the location of the most intense branch of zone NE-1 /Markström and Erlström, 1996/.*

A1.2.3 Fracture set analysis

Fracture orientation data from /Berglund et al. 2003/, which was collected during tunnel mapping within deformation zone NE-1 (between +1,292 and +1,323) has been analysed using stereographic projection methods. The results of the stereographic analyses are presented in Figure A1-3 and Figure A1-4 below.

A summary of the identified fracture sets is presented in Table A1-3 below:

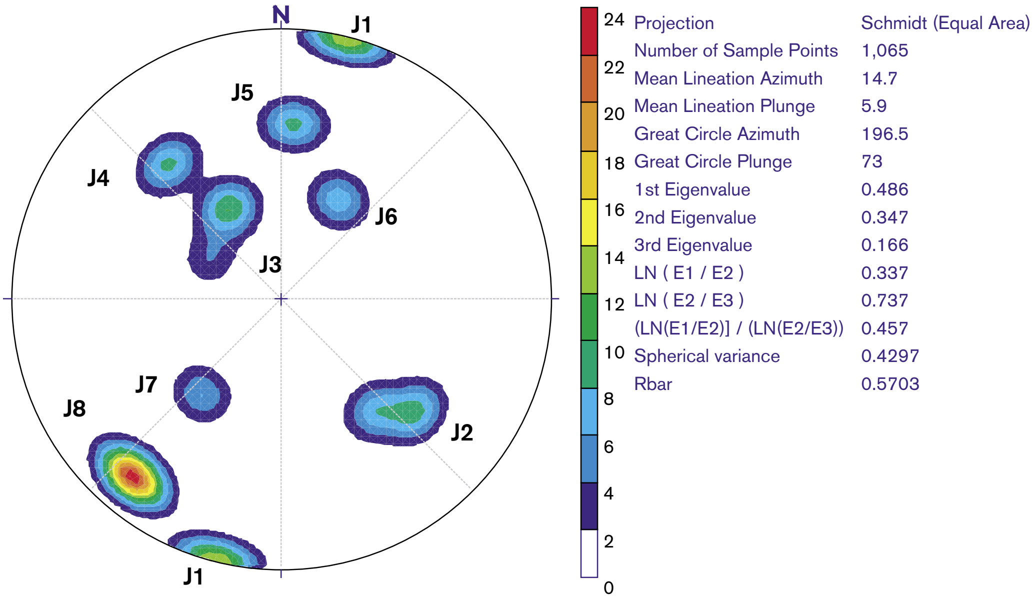

Figure A1-3. *Contour plot of poles to planes for all non-water-bearing fractures mapped for tunnel section +1,292 to +1,323.*

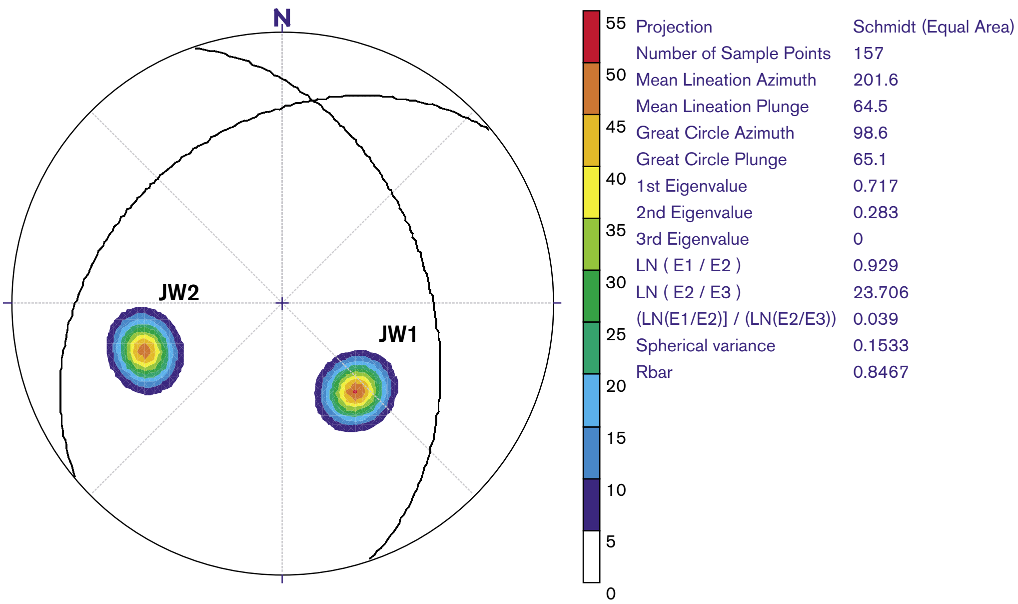

Figure A1-4. *Contour plot of poles to planes with corresponding great circles for all water-bearing fractures mapped for tunnel section +1,292 to +1,323 with the exception of the fractures in the clay core.*

Table A1-3. Summary of the fracture sets identified in the stereographic analysis.

Waterbearing			Non-waterbearing		
Fracture set	Strike	Dip	Fracture set	Strike	Dip
JW1	230 (NE-SW)	35° NW	J1	284 (E-W)	Vertical
JW2	341 (NNW-SSE)	45° ENE	J2	225 (NE-SW)	50° NW
			J3	045 (NE-SW)	30° SE
			J4	050 (NE-SW)	60° SE
			J5	094 (E-W)	60° S
			J6	120 (WNW-ESE)	35° SSW
			J7	310 (NW-SE)	38° NE
			J8	310 (NW-SE)	75° NE

It should be noted that the analysis has not considered the fractures recorded in the 1 m wide central core zone (mapped as Z7 in Figure A1-2), in which 933 waterbearing, clay filled fractures, spaced at 0.002 m were recorded. The reason for this is that these fractures are so closely spaced and so large in number that they mask the structure of the rock mass over the whole deformation zone.

The decision to plot the waterbearing fractures on a separate stereogram was based on the fact that these fractures are of particular importance, due to their impact on the tunnelling process. If these had been plotted on the same stereogram as the non-waterbearing fractures, their presence may have been masked by other joint sets and hence their significance may have been overlooked.

Numerous other fracture systems can be identified from the analysis, aside of the dominant NE-SW trending clay core zone of NE-1. The most prominent non-waterbearing fracture set displayed in Figure A1-3 is J8, which trends NW-SE and dips relatively steeply (approx 75°) to the NE (i.e. perpendicular to NE-1). Another steeply dipping set (J1) trends E-W. Apart from these two steeply inclined fracture sets, the analysis has identified four fracture sets (J3, J4, J5 and J6) that trend between 045–120 and dip between 30–60° S. The combination of these sets, together with sets J2 and J7 suggest that the fracture orientations would be conducive to block instability in the tunnel roof and walls.

The analysis also identified two waterbearing fracture sets, JW1 and JW2. These two sets undoubtedly contributed to further tunnel instability (the 933 fractures in the clay core were logged as waterbearing but have been discounted for reasons previously discussed).

A1.2.4 Discussion of geological information

The NE-SW strike of NE-1 predicted by these studies was shown to be accurate since subsequent studies have recorded a strike of 060 for the deformation zone. However, the anticipated dip (50–60° NW) was 10–20° shallower than that quoted in the earlier reports and 10–25° shallower than that quoted in /Rhén et al. 1997a/. This is not altogether unsurprising as predicting dip and strike for such a large, complex structure based on borehole and geophysical evidence is relatively difficult.

The predicted width of the deformation zone was significantly larger than the width of the deformation zone intersected in the tunnel (50–70 m predicted width against 30 m recorded in the tunnel). This can also be attributed to the complex nature of the deformation zone and the uncertainty associated with borehole and geophysical information. In terms of the post construction reports, the more intensely fractured and deformed zone is in the earlier reports described as being 5 m wide, however, an 8 m wide highly fractured zone is described in /Rhén et al. 1997a/. All the post-construction reports refer to a 10–15 m wide section of more or less fractured rock on either side of the central zone.

Several of the reports refer to the presence of swelling clay in the fault gouge. A clay sample from the deformation zone (distance 1/306) was sent for XRD analysis, which showed that half the clay was of a strongly swelling type /Hamberger and Didriksson, 1992/. However, during the passage of NE-1, no difficulties associated with swelling clay were reported in the SKB's reports.

The analysis of the fracture data from mapping of NE-1 highlights two waterbearing fracture sets (aside of the deformation zone itself). Fracture set JW1 dips moderately to the NW and JW2 trends almost north-south. The fractures comprising set JW2 may be those described as the steeply dipping, highly conductive N-S trending fractures described in /Rhén and Stanfors, 1993/, which are potentially troublesome as they are difficult to identify during probing and may be encountered in long sections of tunnel (near vertical fractures which are parallel with the tunnel). In addition to the waterbearing fractures, the stereonet analysis located eight different non-waterbearing fracture sets, which add to the complexity of the deformation zone.

A schematic plan diagram of the deformation zone has been constructed (Figure A1-5). Figure A1-5 is principally based on the information from the post construction reports, which are interpreted to be the most representative data source for deformation zone NE-1.

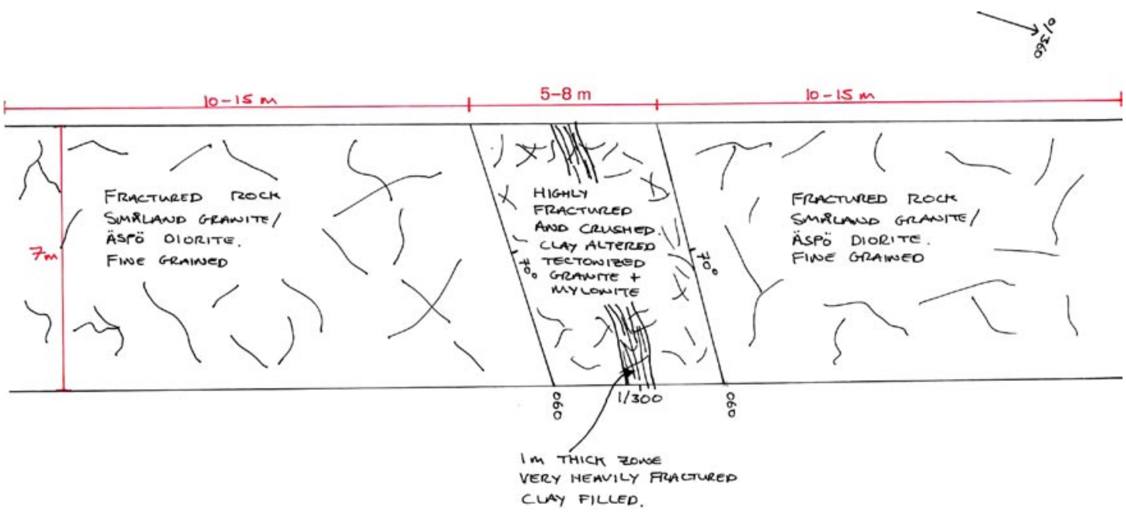

Figure A1-5. Schematic plan of the most intense branch of deformation zone NE-1 (not to scale).

A1.3 Rock mass properties

A1.3.1 General

The objective of this section is to summarize the mechanical properties of the rock mass encountered in deformation zone NE-1. The rock mass properties are used as the basis for the analyses conducted in this study.

The information presented is taken from the following reports:

- PR-HRL-96-07 "Summary of rock mechanical results from the construction of Äspö Hard Rock Laboratory." /Stille and Olsson, 1996/.
- PR-HRL-96-19 "Overview of documentation of tunnel, niches and cored boreholes." /Markström and Erlström, 1996/.
- TN-25-92-19N "Passage of waterbearing fracture zones. Passage of NE-1. Tunnel stability and recommendations for rock support at NE-1." /Stille and Olsson, 1992/.
- R-02-04 "Strategy for a rock mechanics site descriptive model. A test case based on data from the Äspö HRL." /Hudson, 2002/.

A1.3.2 Laboratory tests

The results of laboratory tests conducted on core samples recovered from the Äspö region are given in /Stille and Olsson, 1996/. The tests were conducted on four rock types taken from cores drilled during both the site investigation and construction phase of Äspö HRL. Cores were selected for each rock type from 2–8 different boreholes that were drilled predominantly in the first part of the tunnel /Stille and Olsson, 1996/. The results of the laboratory tests are presented in Table A1-4 (site investigation drillcore) and Table A1-5 (drill core from construction).

No laboratory test results for rock samples coming directly from deformation zone NE-1 have been located in SKB's reports.

Table A1-4. Laboratory test results from site investigation phase /Stille and Olsson, 1996/.

	Greenstone	Fine-grained granite	Äspö diorite	Småland granite
Uniaxial compressive strength				
Mean (MPa)	119	236	184	189
Interval (MPa)	103–168	152–336	164–217	147–260
Young's modulus				
Mean (GPa)	53	65	60	62
Interval (GPa)	32–74	59–70	54–65	62–63
Poisson's ratio				
Mean	0.25	0.22	0.23	0.24
Interval	0.24–0.26	0.20–0.22	0.20–0.25	0.24
Brittleness	More brittle	Less brittle	Brittle	Brittle

Table A1-5. Laboratory tests from construction phase /Stille and Olsson, 1996/.

	Greenstone	Fine-grained granite	Äspö diorite	Småland granite
Number of test	10	9	10	10
Uniaxial compressive strength				
Mean (MPa)	207	258	171	255
Interval (MPa)	121–274	103–329	103–210	197–275
Standard deviation (MPa)	53	78	35	29
Young's modulus				
Mean (GPa)	78	77	73	74
Interval (GPa)	71–96	72–80	65–80	63–79
Standard deviation (GPa)	10	3	4	5
Poisson's ratio				
Mean	0.24	0.23	0.24	0.23
Interval	0.18–0.31	0.21–0.25	0.22–0.29	0.20–0.26
Standard deviation	0.04	0.01	0.02	0.02
Brittleness	Brittle	More Brittle	More Brittle	More Brittle

A1.3.3 Rock mass rating for the Äspö tunnel

The Rock Mass Rating (RMR) system /Bieniawski, 1989/ was used for the classification of the rock mass during the construction of the Äspö HRL /Markström et al. 1996/. This system rates the rock mass using the following six parameters.

1. Uniaxial compressive strength of rock material;
2. Rock Quality Designation (RQD);
3. Spacing of discontinuities;
4. Condition of discontinuities;
5. Groundwater conditions;
6. Orientation of discontinuities.

The predicted (from site investigation) and observed (from construction) RMR values along the whole Äspö tunnel are presented in Table A1-6.

In general, the predicted and outcome values presented in Table A1-6 are in reasonably good agreement. The percentage of rock falling in the very good (class A) to good (class B) categories during construction was slightly less than that predicted (73% predicted, 67% outcome). The percentage of rock mapped in the poor (class D) and very poor (class E) categories during construction was also less than predicted (8% predicted, 4% outcome). These differences are accounted for by the fact that a larger percentage of fair (class C) rock was mapped during construction than was predicted (19% predicted, 28% outcome).

Table A1-7 presents a summary of the observed RMR-values in the Äspö tunnel for the various rock types encountered.

The data in Table A1-7 shows that there is a relationship between RMR values and rock type. Fine-grained granite, which had a high fracture frequency, has a significantly lower mean and larger range of RMR values than the other three rock types.

Table A1-6. Summary of predicted and observed RMR-values along the entire tunnel /Stille and Olsson, 1996/.

Class	RMR-value	Predicted distribution	Outcome distribution
A	> 72	23%	28%
B	60–72	50%	39%
C	40–60	19%	28%
D	< 40, zones < 4 m	3%	4%
E	< 40, zones > 4 m	5%	

Table A1-7. Outcome RMR-values in different rock types for the entire tunnel /Stille and Olsson, 1996/.

	Greenstone	Fine-grained granite	Äspö diorite	Småland granite
Mean	64	48	69	65
Interval	53–74	15–89	28–97	35–92
Std deviation	6	13	10	11
No of values	18	69	289	202

A1.3.4 Rock mass rating for NE-1

A predication of the rock mass classes for deformation zone NE-1 was made prior to construction of Äspö HRL and is presented in Table A1-8 below.

During the tunnel excavation, RMR-values of deformation zone NE-1 were determined by geological mapping. The RMR values between chainage 1/285 and 1/320 are presented in Figure A1-6 as well as Table A1-9 /Markström and Erlström, 1996/. A frequency distribution of these RMR-values is given in Table A1-10.

Table A1-8. Predicted distribution of rock mass classes in NE-1 /Stille and Olsson, 1992/.

RMR	Diorite	Aplite	Sum
60–72	40%	5%	45%
40–60	15%	10%	25%
< 40 (zones less than 4 m width)	10%	20%	30%
Sum	65%	35%	100%

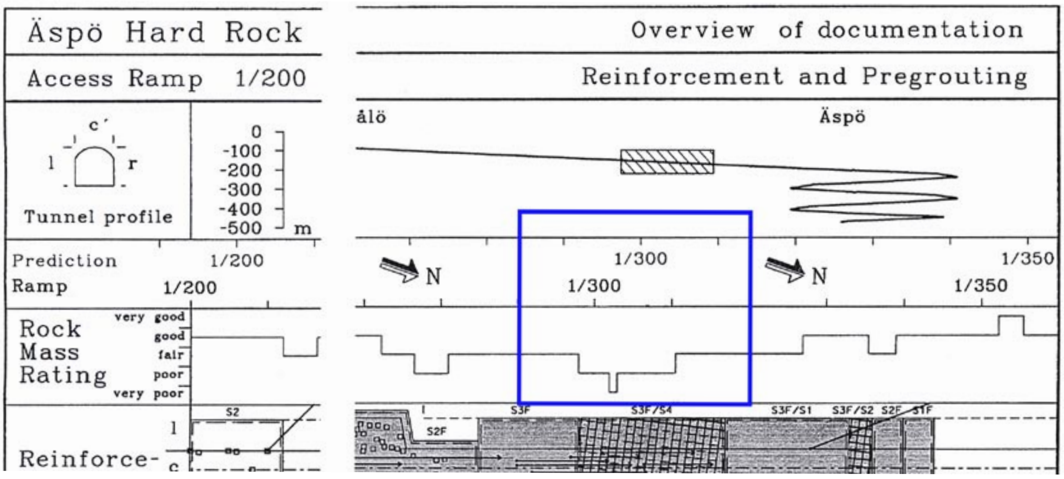

Figure A1-6. Geological mapping of NE-1 /Markström and Erlström, 1996/.

Table A1-9. RMR-values for deformation zone NE-1 from tunnel mapping /Markström and Erlström, 1996/.

Chainage	Description	RMR
1/285–1/298	Fair	41–60
1/298–1/301	Poor	21–40
1/301–1/303	Very poor	< 21
1/303–1/310	Poor	21–40
1/310–1/320	Fair	41–60

Table A1-10. Frequency distribution of RMR-values mapped in deformation zone NE-1 based on data presented in Table A1-9.

Description	RMR	Tunnel length for each RMR interval
Fair	41–60	23 m
Poor	21–40	10 m
Very poor	< 21	2 m

A1.4 *In situ* rock stress

A1.4.1 General

An extensive stress measurement programme has been carried out by SKB in order to gain an understanding of the *in situ* rock stresses. This section is intended as a summary of the findings of these measurements and is based on the following three reports:

- PR-HRL-96-07 "Summary of rock mechanical results from the construction of Äspö Hard Rock Laboratory." /Stille and Olsson, 1996/.
- R-02-03 "Strategy for a rock mechanics site descriptive model. Development and testing of an approach to modelling the state of stress." /Hakami et al. 2002/.
- R-02-35 "Simpevarp – Site description model version 0." /SKB, 2002a/.

A1.4.2 Rock stress measurements

Hydraulic fracturing tests were conducted in three boreholes drilled from the surface (KAS02, KAS03 and KAS05) prior to construction of Äspö HRL. The measurements indicated that the maximum horizontal stress was oriented NW-SE. *In situ* stress measurements performed within the depth range 300 to 500 m suggested that the ratio between the maximum horizontal stress and the vertical stress, K_0, was 1.7 /Stille and Olsson, 1996/.

A stress measurement programme conducted in boreholes was also undertaken during the tunnel construction. The results from this programme also indicated that the dominant horizontal stress was oriented in a NW-SE direction.

The Simpevarp site descriptive regional model version 0 /SKB, 2002a/ uses the *in situ* stress magnitudes and orientations shown in Table A1-11 and Table A1-12 respectively.

Using the relationships presented in Table A1-11, the magnitudes of the principal stresses at depths of 200 m, 400 m and 600 m have been calculated and the results are presented in Table A1-13. There values will be the base for the stresses to be used in the descriptive model for this study.

According to the orientations of the principal stresses given in Table A1-12, it is reasonable to assume that:

$\sigma_1 = \sigma_H$ (major horizontal principal stress),

$\sigma_2 = \sigma_v$ (vertical principal stress),

$\sigma_3 = \sigma_h$ (minor horizontal principal stress).

Table A1-11. *In situ* stress magnitudes in the Simpevarp regional model area /SKB, 2002a/.

Parameter	σ_1	σ_2	σ_3
Mean stress magnitude (MPa)	0.066z+3	0.027z	0.022z+1
Uncertainty, 0–500 m	± 25%	± 25%	± 25%
Uncertainty, 500–2,000 m	± 40%	± 25%	± 40%
Spatial variation, rock mass	± 15%	± 15%	± 15%
Spatial variation, fracture zones	± 50%	± 50%	± 50%

Table A1-12. *In situ* stress orientations in the Simpevarp regional model area /SKB, 2002a/.

Parameter	σ_1, trend	σ_1, plunge	σ_3, trend	σ_3, plunge
Mean stress orientation	133°	0°	43°	0°
Uncertainty, 0–500 m	± 15°	± 10°	± 15°	± 15–45°[1]
Uncertainty, 500–2,000 m	± 30°	± 10°	± 30°	± 10°
Spatial variation, rock mass	± 15°	± 15°	± 15°	± 15°
Spatial variation, fracture zones	± 25°	± 30°	± 25°	± 30°

[1] At some level, σ_2 and σ_3 may have similar magnitude and the dip can become undefined.

Table A1-13. Stress magnitudes at 200, 400 and 600 m depth.

Depth (z) (m)	σ_1 (MPa)	σ_2 (MPa)	σ_3 (MPa)
200	16.2 ± 4.1	5.4 ± 1.4	5.4 ± 1.4
400	29.4 ± 7.4	10.8 ± 2.7	9.8 ± 2.5
600	42.6 ± 17.0	16.2 ± 4.1	14.2 ± 5.7

A1.5 Hydrogeological description of NE-1

A1.5.1 General

The objective of this section is to summarize the hydrogeological data that relates to deformation zone NE-1. This data is used as the basis for the hydrogeological component of the descriptive model in Chapter 2 and is based on information taken from the following reports:

- PR-25-92-18 "Passage through water-bearing fracture zones. Evaluation of investigations in fracture zones NE-1, EW-7 and NE-3." /Rhén and Stanfors, 1993/.
- PR 25-92-18A "Passage through water-bearing fracture zones. Investigations during passage of fracture zone EW-7 and NE-3." /SKB, 1992a/.
- PR 25-92-18D "Passage through water-bearing fracture zones. Construction and grouting." /SKB, 1992b/.
- TR 97-05 "ÄSPÖ HRL. Geoscientific evaluation 1997/4. Results from pre-investigations and detailed site characterization. Comparison of predictions and observations. Hydrogeology, groundwater chemistry and transport of solutes." /Rhen et al. 1997b/.
- TR-97-06 "ÄSPÖ HRL. Geoscientific evaluation 1997/5. Models based on site characterization 1986–1995." /Rhen et al. 1997a/.

A1.5.2 Flow measurements

Deformation zone NE-1 was carefully characterized using a variety of hydrological testing methods during the site investigation and construction phases of the Äspö tunnel. Figure A1-7 shows the investigation boreholes that were drilled from the tunnel prior to entering deformation zone NE-1. Flow rates were estimated for sections of each borehole drilled from the tunnel. The results from these measurements are presented in Table A1-14. A flow spin metre was also used to measure flow rates in core borehole KA1061A (Table A1-15). It can be seen in Figure A1-7 that most of the tests were done in the southern part of NE-1.

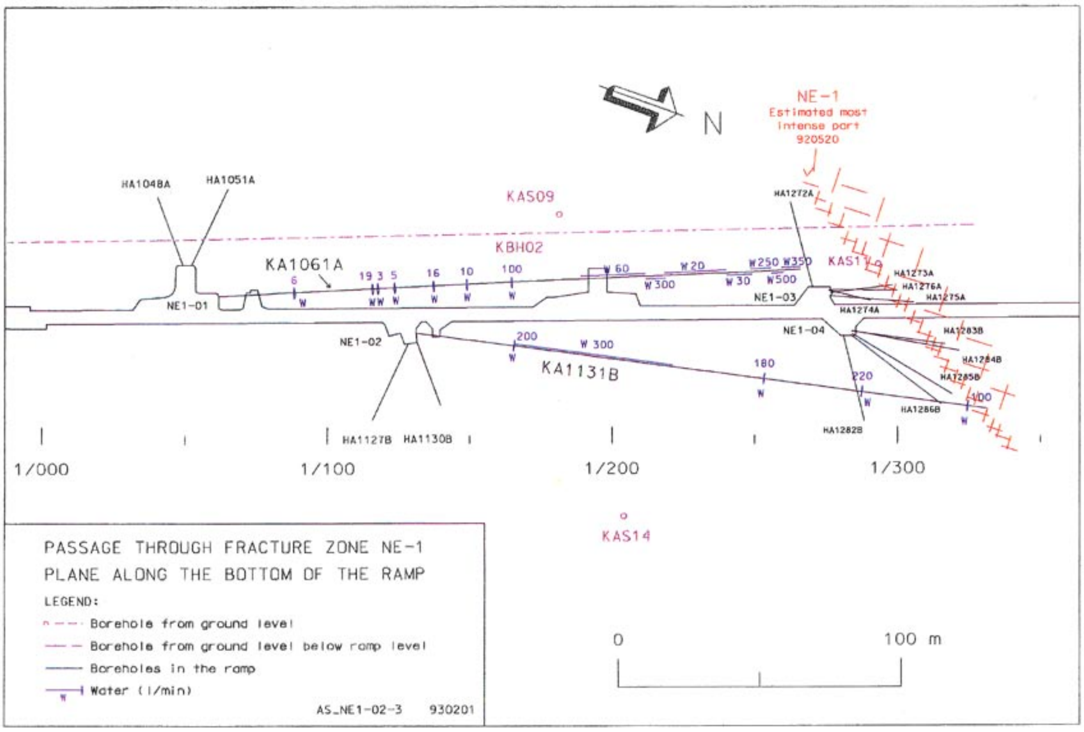

Figure A1-7. Percussion boreholes and core boreholes drilled from the tunnel, for conducting hydrogeological tests /Rhen and Stanfors, 1993/. For a more detailed diagram of the holes drilled in proximity to deformation zone NE-1 see Figure 2-2, Chapter 2.

Table A1-14. Flow rates for boreholes drilled into deformation zone NE-1 /Rhen and Stanfors, 1993/.

Borehole	Depth of borehole (m)	Total amount of water (l/min)	Distribution of flow rate (l/min)
HA1272A	32	150	100 at 29 m 150 at 32 m
HA1273A	23	1,380	100 at 14 m 400 at 17 m 500 at 20 m the rest is distributed along the bottom part of the borehole
HA1274A	18	2,000	2,000 at 16 m
HA1275A	29.8	1,600	250 at 26 m 1,000 at 29 m 1,600 at the bottom of the borehole.
HA1276A	24.5	480	380 at 22 m the rest is distributed along the bottom part of the borehole
HA1282B	31.5	27.6	20 at 10 m 24 at 23 m the rest is distributed along the bottom part of the borehole
HA1283B	35.5	1,350	200 at 31 m the rest is distributed along the bottom part of the borehole
HA1284B	38.5	270	20 at 24 m 270 at 31 m
HA1285B	41.5	0	–
HA1286B	40.5	33	10 at 16 m 20 at 22 m 30 at 34 m

Table A1-15. Flow measurements in KA1061A. Estimated inflow rates to the borehole /Rhen and Stanfors, 1993/.

	Conductive structure	
Estimated flow rate from spinner or drilling (l/min)	Estimated from drilling record Depth of borehole (m)	Estimated from spinner Depth of borehole (m)
6		27
19		55
3		57
5		63
16		77
10		89
100	105	
60	130–153	
300	153–160	
20	160–182	
30	182–191	
250	191–198	
500	198–202	
350	202–208.5	

A1.5.3 Transmissivity (prior to grouting)

The transmissivity of deformation zone NE-1 was estimated by conducting hydraulic testing in probing boreholes that intersected the deformation zone.

Evaluation of the data obtained by interference testing in the percussion boreholes with HA1273A (freely flowing), was conducted by /Rhén and Stanfors, 1993/. The evaluation results are given in Table A1-16 and the hydraulic data in Table A1-17. The results represent the transmissivity obtained by the tests prior to the grouting operations.

A1.5.4 Effects of grouting on transmissivity

When passing through the deformation zone, pre-grouting was conducted in order to lower the rate of water into the tunnel. Re-grouting was performed in places where the tunnel was not sufficiently watertight. In each grouting fan, typically 10–25 grouting boreholes were drilled, tested for water loss and grouted /Bäckblom et al. 1994/. The effect of grouting on the transmissivity of the deformation zone has been evaluated by measuring water inflow from the grout boreholes. Only the effects of the first grouting cycle for each tunnel section have been evaluated.

Median transmissivity was calculated from flow measurements conducted in boreholes at various sections of the tunnel after grouting. The results of these measurements are shown in Table A1-18 and a graph of this data is presented in Figure A1-8. The measured transmissivity in the first fan of NE-1 (fan no 65, section 1/290), resulted in a higher transmissivity than the following measurements. This is most likely due to the fact that it was the first grouting fan in unaltered rock (later fans benefited from the effects of grouting conducted in the earlier fans).

Table A1-16. Transmissivity of NE-1 evaluated by /Rhén and Stanfors, 1993/.

Transmissivity of NE-1, evaluation based on:	Range	Geometric mean
Early time response	4×10^{-5}–1×10^{-3} m^2/s	4.5×10^{-4} m^2/s
Late time response	8×10^{-5}–7×10^{-4} m^2/s	4.0×10^{-4} m^2/s

Table A1-17. Summary of hydraulic data from interference test in HA1273A at tunnel depth /Rhen and Stanfors, 1993/.

Borehole	T (m^2/s)	R (m)	S (–)	Recovery after dt, = 10 min (m)	Comments
HA1272A : JA	4.4×10^{-4}	45	6.8×10^{-4}	14.7	Radial flow 30 s–
HA1273A : JB	4.9×10^{-4}	0	–	164	ξ = 13
HA1274A : JC	4.4×10^{-4}	9	1.0×10^{-4}	17.0	Lin 1 min, r∂d fl 1 min
HA1275A : JD	4.7×10^{-4}	15	3.0×10^{-5}	17.2	Radial flow 30 s–
HA1276A : JE	4.9×10^{-4}	12	4.3×10^{-5}	17.4	Radial flow 30 s–
HA1282B : JF	4.4×10^{-4}	32	2.6×10^{-5}	12.4	Radial flow 30 s–
HA1283B : JG	4.2×10^{-4}	36	5.6×10^{-6}	20	
HA1284B : JH	–	30	–	30	
HA1285B : JI	8.1×10^{-4}	46	2.7×10^{-5}	7.0	Radial flow/5 min Linear 5 min
HA1286B: JJ	4.7×10^{-4}	33	8.5×10^{-6}	16	Radial flow 30 s–
KA1061A : F	4.3×10^{-4}	45	5.2×10^{-6}	16	Linear 0–1 min Radial 1 min
KA1131B : I	4.3×10^{-4}	45	2.0×10^{-5}	11	Radial flow 30 s–
KBH02 : CI	4.3×10^{-4}	40	6.2×10^{-6}	16	Radial flow 30 s–
KAS09 : MA94	–	105	–	1.15	
KAS09 : MA93	–	112	–	1.30	
KAS09 : MA92	–	150	–	0.5	
KAS11 : MA113	4.4×10^{-4}	39	1.0×10^{-5}	14.5	Lin 0–3 min, rad fl 3 min
KAS11 : MA112	3.0×10^{-4}	13	1.9×10^{-4}	15.6	Linear 0–3 min Radial 3–10 min Bilinear? 10–
KAS11 : MA111	4.4×10^{-4}	44	1.4×10^{-5}	11.7	Radial flow–10 min, Bilinear? 10 min–
KAS14 : MA145	–	120	–	1.0	
KAS14 : MA144	–	109	–	1.0	
KAS14 : MA143	–	109	–	1.0	
KAS14 : MA142	–	110	–	1.0	

Table A1-18. Median transmissivities after grouting for different sections of NE-1 /Stille et al. 1993/.

Fan no	Section (m)	Median transmissivities (m^2/s)
65	1/290	18.0×10^{-6}
71	1/292	0.3×10^{-6}
75	1/294	1.3×10^{-6}
84	1/297	3.3×10^{-6}
89	1/299	4.0×10^{-6}
91	1/301	4.5×10^{-6}
93	1/306	2.5×10^{-6}

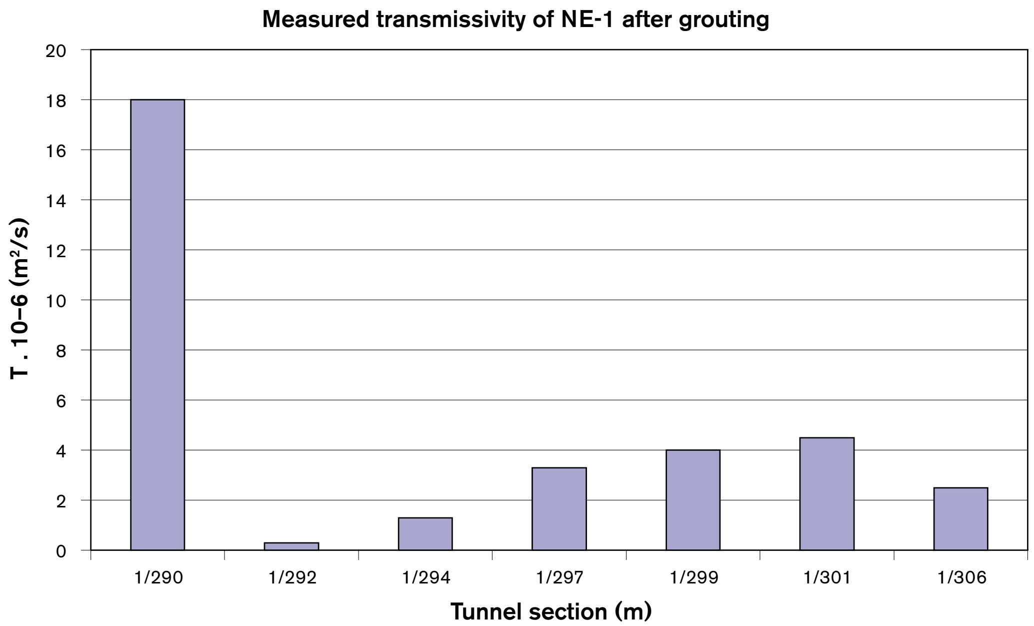

Figure A1-8. *Median transmissivity after grouting of NE-1 as measured at each tunnel section during tunnel construction (based on data in Table A1-18).*

A comparison of the transmissivities measured for the entire deformation zone prior to grouting (Table A1-16), and for the various sections after grouting (Table A1-18), suggests that the transmissivity of deformation zone NE-1 is reduced by an order of 10^{-2} m/s^2 due to the first grout process. Re-grouting of sections that were deemed to be insufficiently watertight would have further reduced the transmissivity, however, no results were reported in the reports supplied by SKB.

A1.5.5 Variation in hydrogeological conditions with depth

Hydraulic tests have been performed in deep core boreholes at Äspö. These tests have been conducted using 30 m and 100 m sections that were isolated with packers. The purpose of the tests was to measure the effective hydraulic conductivity of the rock mass and its dependency on depth. A depth versus hydraulic conductivity graph based on these tests is presented in Figure A1-9, which indicates that there is a crude relationship between depth and hydraulic conductivity. However, the scatter in the data is considerable, particularly

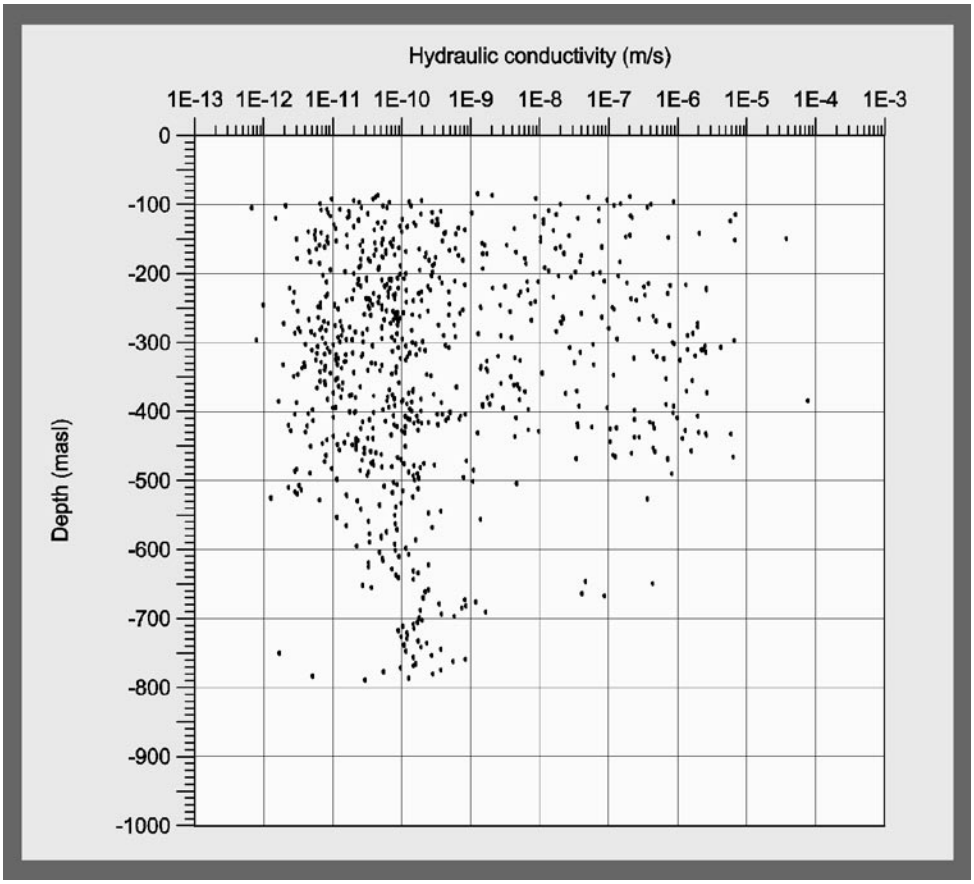

Figure A1-9. *Effective conductivities measured at Äspö /Rhen et al. 1997b/.*

in the upper 500 m. Below 500 m there is a general decrease in hydraulic conductivity, however, all the data taken below 500 m is based on only one vertical borehole and should be treated with care. What can be concluded is that the assumption that hydraulic conductivity will decrease with depth is rather general.

It should be considered that there is no detailed information relating to the variation of hydraulic conductivity with depth in deformation zones such as NE-1.

A1.5.6 Other hydrogeological parameters

Porosity

To estimate the effective flow porosity of NE-1 at tunnel depth, two tracer tests were conducted. The tracer fluid was injected in boreholes KAS14 and KAS09 and recovered in KA1131B. The interpretation of the tracer tests was that deformation zone NE-1 consists of many interconnected fracture flow-paths of moderate to high hydraulic conductivity. The mean flow porosity was estimated at 0.007 with a range of 0.06%–2.2% /Rhén et al. 1993/.

Videoscope investigations were conducted in order to assess the porosity of the rock mass after grouting. Results from these tests suggest that the porosity of the rock mass is between 0.29–0.35% /Rhén et al. 1993/.

Skin factor

The skin factor for boreholes KA1061A and KA1131B has been estimated from interference tests conducted before and after the tunnel was constructed. The results are summarized in Table A1-19 /Rhén et al. 1993/.

Grout filled fracture aperture

The thickness of grout filled fractures from 1 mm up to 50 mm was evaluated from the results of the videoscope investigations. The frequency distribution of the thickness of grout-filled fractures from all boreholes is shown in Figure A1-10 /Rhén et al. 1993/.

Table A1-19. Skin factors obtained from tests in KA1061A (1) and KA1131B (2) /Rhén et al. 1993/.

KA1061A (1)		
Parameter	**January 1992 test**	**November 1992 test**
Flow rate, Q	1.61×10^{-2} m³/s	1.67×10^{-2} m³/s
Transmissivity (0.1–0.5 min)	8.4×10^{-4} m²/s	8.7×10^{-4} m²/s
Skin factor (0.1–0.5 min)	33	30
Transmissivity (1–40 min)	4.8×10^{-4} m²/s	4.5×10^{-4} m²/s
Skin factor (1–40 min)	17	12
KA1131B (2)		
Parameter	**February 1992 test**	**November 1992 test**
Flow rate, Q	6.13×10^{-3} m³/s	5.27×10^{-3} m³/s
Transmissivity (0.05–0.2 min)	$4.0^{-5}\times10$ m²/s	$4.3^{-5}\times10$ m²/s
Skin factor (0.05–0.2 min)	–1.8	–1.2
Transmissivity (0.5–1.5 min)	6.5×10^{-5} m²/s	2.5×10^{-5} m²/s
Skin factor (0.5–1.5 min)	2.7	–4
Transmissivity (8–40 min)	3.7×10^{-4} m²/s	2.5×10^{-4} m²/s
Skin factor (8–40 min)	50	33

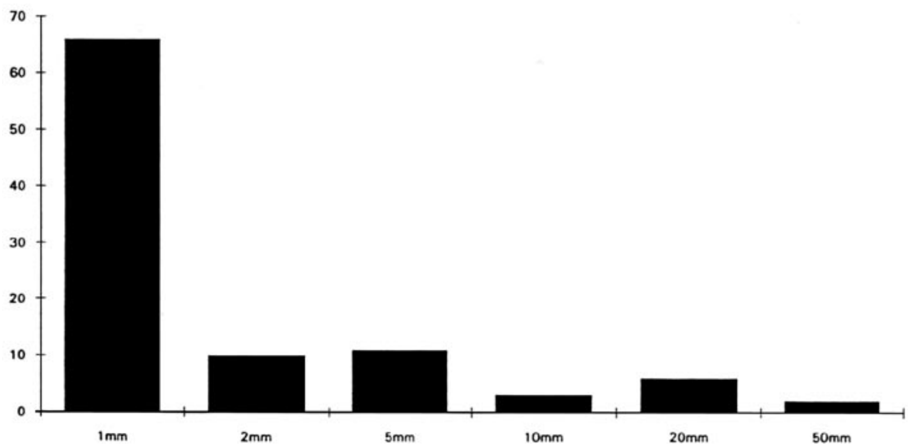

Figure A1-10. *Thickness frequency distribution of grout-filled fractures /Rhén et al. 1993/.*

A1.6 Experience gained from tunnel construction

Characterization of deformation zone NE-1 prior the entry had played an important role in the tunnelling operation. Investigation drillings were conducted both from the surface and within the tunnel. A summary of the drilling program is presented in Table A1-20.

To complement the three core holes drilled during the pre-investigation phase, four new percussion holes were drilled from the ground surface in order to further characterize the deformation zone prior to driving the tunnel.

The first phase of the passage of deformation zone NE-1 involved additional investigation of the deformation zone. Two cored boreholes were drilled from niches sited approximately 200 m in front of the predicted location of deformation zone NE-1. From each niche, two percussion holes were also drilled to detect possible N-S trending water-bearing fractures. Borehole radar was used to try to gain information on the large-scale orientation of the deformation zone and spinner measurements were used to measure flow rates from water-bearing fractures.

After phase 1, the tunnel was driven closer to the deformation zone and a second phase of investigation drilling was conducted to locate the boundary of NE-1 more accurately. Phase two included the drilling of eight percussion holes (26–40 m) from two niches just in front of NE-1. An additional two percussion boreholes were also drilled from each niche to detect possible N-S trending water-bearing fractures. Borehole investigations from the second phase of drilling included /Rhén et al. 1993/:

- Flow measurements (inflow);
- Radar measurements using RAMAC;
- Interference tests;
- Tracer tests;
- Videoscope investigations to estimate rock porosity;

A sample of clay at section 1/306 was sent for WRD-analysis. The results indicated that half of the clay content was of a strongly swelling type /Hamberger et al. 1992/.

Table A1-20. Summary of drilling programme for NE-1.

Programme	Drillhole
Core holes drilled from the surface	KAS 08 KAS 09 KBH 02
Percussion holes drilled from the surface	LMJ 01 HAS 21 HLX 08 HLX 09
Phase 1 – Drilling from tunnel	KA1061A drilled at 1/050 KA1131A drilled at 1/130 Percussion holes
Phase 2 – Drilling from tunnel	Percussion holes

A1.6.1 Drilling operations

Problems associated with the drilling were encountered due to extremely high water pressures. The drill rods were pushed out by the water pressure at such high speeds that the safety of operators was of concern. Moreover, a large amount of water was emitted from the boreholes leading to hazardous working conditions (see Figure A1-11).

To cope with the high water pressures, a special arrangement using a drill niche, pump niche, borehole casing and a water valve was developed (Figure A1-12). This arrangement meant that the drill site was relatively dry during drilling. When the drill string was extracted, the valve was closed in order to limit the inflow of water. The system also improved the possibility to measure flow rates from the borehole /Rhén et al. 1993/.

Figure A1-11. *Extreme water pressures caused inflows of up to 1,350 l/min in certain drillholes /Rhén et al. 1993/.*

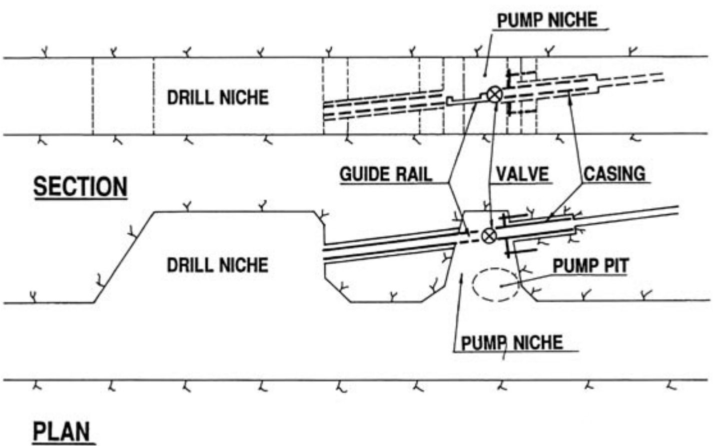

Figure A1-12. *Special drilling arrangement with drill niche, pump niche and water valve /Rhén and Stanfors, 1993/.*

A1.6.2 Grouting activities

The grouting process in the Äspö tunnel followed traditional Swedish practice. This involved drilling a fan of 10–25 holes that were systematically tested for water loss using borehole packers. The data from the water loss measurements was used to estimate the required grout quantity and pressure prior to grouting.

The hydraulic properties of the deformation zone posed serious problems for the grouting operations and were a major reason for the slow rate of tunnel advance. According /Rhén and Stanfors, 1993/ the following difficulties were encountered:

- The limit criteria of grouting volume was set too low at the beginning, so that the grouting had very little sealing effect. When the limit was lifted, the grouting process produced much better results.
- The grouting materials were not stable which caused the cement to be washed out by the water.
- The hardening time for the cement grout was initially too long. This was one of the reasons for the initial low productivity of the grouting operation.

In the early stages of the passage of NE-1 numerous different grout mixtures were tested with varying degrees of success. Results of tests conducted at the Swedish Royal Institute of Technology in Stockholm suggested that a cement based grout mix using CaCl as an accelerator could improve the grouting process. The mixture was tested in the latter stages of the passage of NE-1 and proved to be successful. As a result, it was used for all remaining grouting within deformation zone NE-1.

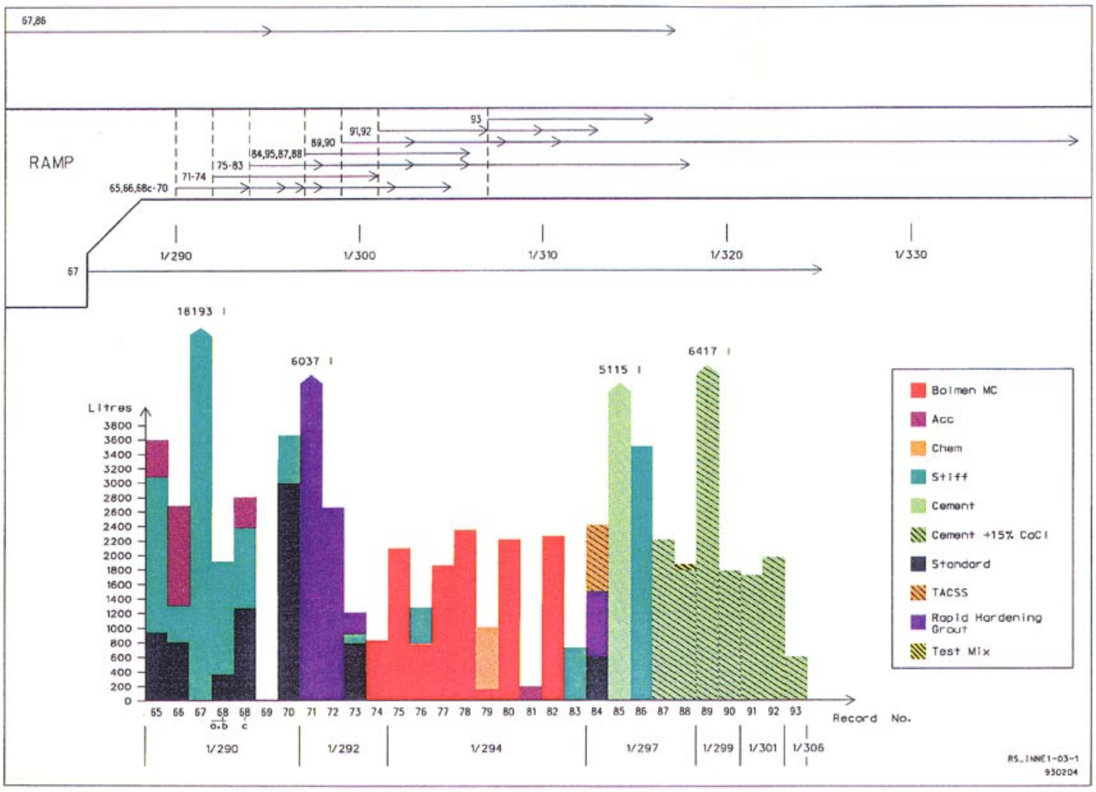

Figure A1-13. Summary of the grouting takes /Rhén et al. 1993/.

Analysis of the grouting results /Stille and Olsson, 1993/ showed that grouting fan no 65 produced the best result. This was thought to be due to the fact that long grouting holes were used to penetrate the water-bearing deformation zone. Grouting within section 1/290 (grout fan 65, 66, 67, 68 69 and 70), including the long percussion holes, accounted for almost 50% of the total grout take. Figure A1-13 shows the grout take and different grout materials used along the tunnel drive. 43% of all grouting was carried out as re-grouting /Stille and Olsson, 1993/. Some details relating to the amount of grouting works required are presented in Table A1-21.

Table A1-21. Grouting statistics for NE-1 based on data from /Stille et al. 1992/.

Parameter	Quantity
Grouted volume	86,000 l
Drilled hole length	6,179 m
Grouted volume per metre of tunnel	3,780 L/m
Drilled hole length per metre of tunnel	270 m/m

A1.6.3 Rock support

Passage of deformation zone NE-1 was relatively straightforward from a stability perspective. The rock support consisted of fibre-reinforced shotcrete, temporary and permanent rock bolts and steel mesh. The support measures designed for the passage of deformation zone NE-1 are shown in Figure A1-14. The permanent rock support consisted of pattern bolting with L = 4 m and c/c = 1.35 m and shotcrete with about 20 cm thickness /Hamberger et al. 1992/.

According to /Stille et al. 1992/ that it is not practical to design the shotcrete lining for full hydrostatic pressure. Therefore measures have to be taken to reduce the water pressure acting the shotcrete lining. The shotcrete must be provided with satisfactory drainage system, or alternatively systematically punctured. The operation of the drainage system must be checked regularly /Stille et al. 1992/. A such drainage system was provided at NE-1 /Rhén et al. 1993/.

A1.6.4 Advance rate

One of the major aims for the passage of deformation zone NE-1 was to gain as much information concerning the hydrogeological regime as possible. The initial concept was, therefore, to attempt to limit the amount of grouting and conduct hydrogeological research tests during the tunnelling process. In the early stages of the passage, this philosophy was applied and, as a result, the advance rate was extremely slow (see Figure A1-15). Between the middle of March 1992 and the middle of May 1992 only two blasting rounds were fired. During mid May engineers abandoned the initial strategy and opted for a new one in which a significant increase in the grout quantity would be implemented and the tunnel would be driven in the most effective manner possible. This resulted in the advance rate increasing dramatically to 1.6 m/day (see Figure A1-15).

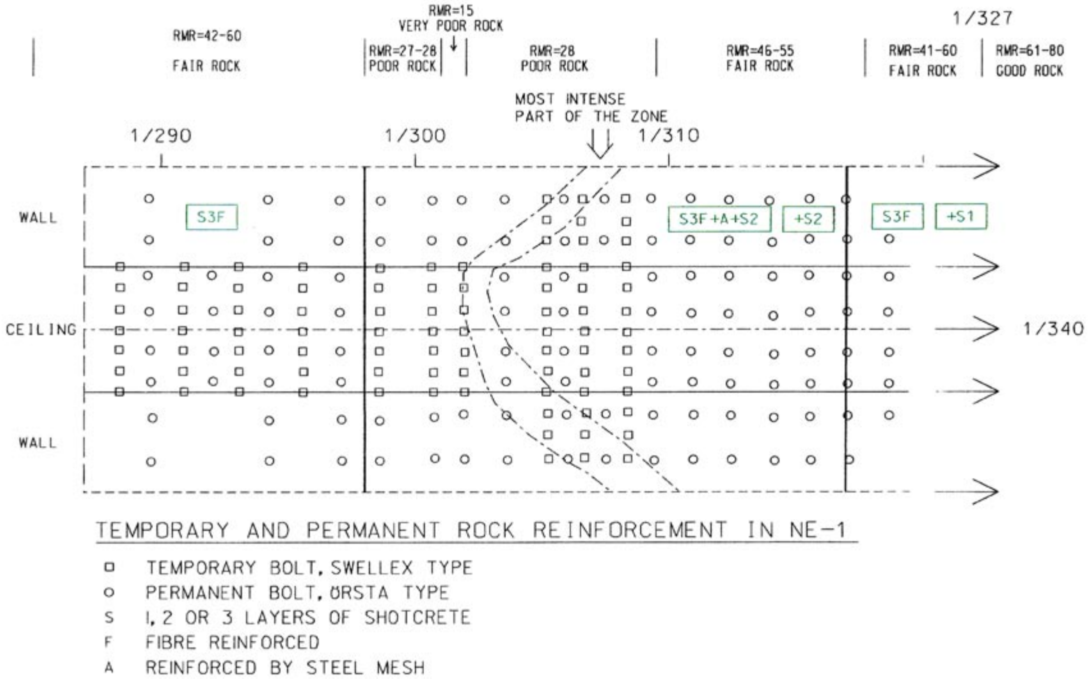

Figure A1-14. *Rock support for NE-1 /Rhén et al. 1993/.*

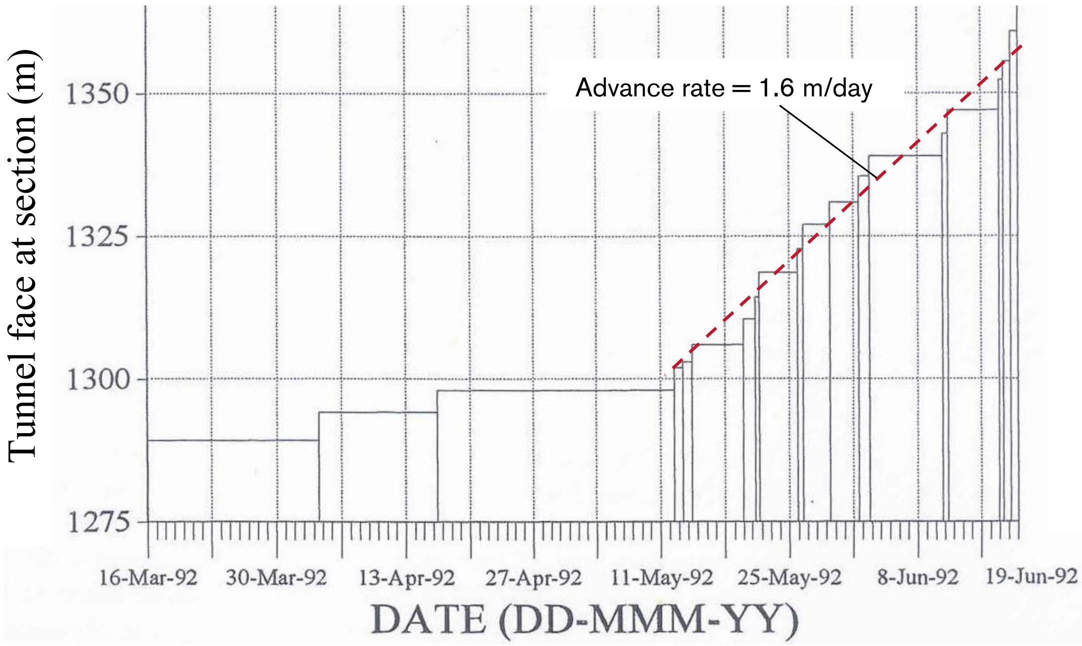

Figure A1-15. *Tunnel advance rate within deformation zone NE-1 /Rhén et al. 1993/.*

A1.6.5 Experiences from ÄSPÖ-HRL

The passage of NE-1 during the construction of the access tunnel for the Äspö HRL has provided a wealth of information on the process of driving a tunnel through a water-bearing deformation zone. A summary of the experiences found in SKB's reports is presented as follows:

- At high water flow, it is found to be expedient to carry out rough sealing first with a few boreholes before the rest of the fan is drilled and grouted /Bäckblom et al. 1994/.
- Long boreholes that penetrate the whole water-bearing zone were very effective for grouting and enabled the entire deformation zone to be mapped /Stille et al. 1993b/ and /Bäckblom et al. 1994/.
- Both theoretical and practical evidence confirmed that the grouting pressure must exceed twice the water pressure to get a good refusal and prevent fingering of the grout. This tended to cause an additional problem as the packers began to creep out /Bäckblom et al. 1994/.
- Grout holes injected with high water-cement ratios did not have the desired sealing effect /Stille et al. 1993b/.
- The high water pressure increases the demands for s quicker hardening of the groute /Stille et al. 1993b/. The use of "Stabilo grout" with a mix of cement, bentonite, plasticizer and silicate increased the initial shear strength of the grout when in a "fluid" state, which can shorten the time for removal of the packers /Bäckblom et al. 1994/.
- The use of calcium chloride as an accelerator improved the sealing effect, however, long-term durability of the grout when using this accelerator is a concern /Bäckblom et al. 1994/.
- Flushing out the gouge material in the zone could increase the effectiveness of the grouting (jet washing) /Bäckblom et al. 1994/.
- The tunnel face should have enough distance to the zone in order to have sufficient stability /Rhén et al. 1993/.
- Geophysical measurements in boreholes give valuable supplementary information, especially regarding the orientation of a structure /Rhén et al. 1993/.
- It is very important to pay attention to hydraulic conductors with a possible orientation more or less parallel to the tunnel. Different hydraulic tests are necessary to confirm the orientation and character of identified hydraulic conductors /Rhén et al. 1993/.
- A special drilling arrangement using a special type of casing with a valve arrangement was used to cope with the high water pressures /Rhén et al. 1993/.

Appendix 2

A2 Case histories

A2.1 General

The ground conditions encountered in every tunnel project are unique. However, similar patterns of rock mass behaviour are witnessed in tunnelling projects throughout the world. As a result, a large amount of information can be obtained by studying such projects. This chapter is intended as a review of the published literature that is thought to have relevance to this study. No direct analogue to the tunnel conditions encountered in deformation zone NE-1 was found in the literature, however, a large number of projects that encountered similar ground conditions or difficulties to those described from the passage of NE-1 (see appendix 1) have been identified. In some cases the geological conditions differ, for example certain tunnel projects driven through sedimentary rocks have been included, principally because whilst the geology is significantly different, many of the problems and methods for overcoming them are of relevance to this project.

The case histories have been tentatively grouped according to the ground conditions and problems encountered in each project. Much of the information presented in this appendix is taken directly from the source. Extracts of text that are taken directly from a publication without modification are therefore presented in *italic type*.

A2.2 Water-bearing weakness zones

A2.2.1 Oslofjord tunnel, Norway

The Oslofjord crossing, located 40 km south of Oslo was a major tunnelling project that included 26.5 km of new highway. The rock types on both sides of the fjord are Precambrian gneisses, which are a common tunnelling medium in Norway. The following extract on the Oslofjord tunnel is taken from /Backer and Blindheim, 1999/.

Seismic investigations identified three major weakness zones, the worst of which was the "Hurum weakness zone". Further investigation of the Hurum weakness zone in the form of two core holes and seismic tomography showed that the material in the central part consisted of crushed rock and clay.

A combination of percussion probe drilling ahead and above the tunnel face and core drilling ahead of the face was employed as the zone was approached from the west. The percussion probe drilling showed that the weakness zone in the upper part of the cross section contained soil materials with a mixture of gravel and sand to clay under full water pressure of 120 m (12 bars). Because of the extensive investigations ahead of the tunnel face, the excavation was stopped in fair rock conditions at a safe distance of 17 m before the zone.

Core holes from the preinvestigations showed better rock conditions in the weakness zone at greater depth. A change of the tunnel alignment was not possible, however, to minimise any delays a bypass tunnel was constructed 15–20 m below the main tunnel.

Further investigation of the zone, particularly the so-called "soil zone" were performed in the form of core drilling. Drilling was found to be difficult due to the high water pressures and in some cases three different casing diameters were required. Two possible methods of stabilisation were considered, stabilisation by grouting and stabilisation by freezing.

Because of the high water pressure, normal cement grouting was considered necessary as a first step for both methods. The main uncertainty was whether further grouting could provide sufficient stability in the soil masses, or if freezing would be required to obtain necessary safety during excavation.

The first phase of cement grouting successfully tightened the crushed rock zone. In the soil zone above, the grouting gradually reduced water leakage to 10–100 l/min per drillhole. Drilling problems made it impossible to drill more than 21 to 25 m in to the soil zone. The presence of the bypass tunnel meant that supplementary grouting from the "back side" of the zone could be conducted.

700 tons of cement grout, including 25 tons of micro-silica and 40 tons of sand were pumped into the weakness zone. In addition, 11 m^3 of concrete were also pumped in as a trial; however, this process was discontinued due to practical reasons.

Continued drilling problems suggested that it would be difficult to stabilise the soil zone. Large volumes of grouting material would be needed and the presence of conductive zones meant that controlling the location of the grout would be difficult. As a result, ground freezing prior to excavation was selected as a more viable alternative. The decision was based primarily on the assumed higher and controllable safety of the stabilisation by freezing.

The chosen medium for the freezing process was salt water. Nitrogen freezing was considered, but was not selected, as the necessary amount of liquid nitrogen would be too difficult to handle safely inside the tunnel. Initial design was based on a single row of freezing pipes spaced at one metre in the soil zone and two metres in the crushed rock zone. Testing at the Norwegian University of Science and Technology showed that for the sand fraction containing salt sea water, a temperature of –28°C was needed in a thickness of 2–3 m to gain sufficient stability while excavating. As a result, a second row of freeze pipes 2–3 m inside the first row was deemed necessary to provide an arch of –28°C.

Drilling of the large diameter holes for the freeze pipes was very time consuming (drilling was estimated to take 1 year) in the soil zone due to the high water pressure and unstable drillholes. The high water pressure also hindered the collection of representative samples of the soil material in the zone. As a result, the design of the freezing operation was based on tests performed on the assumed worst-case material of single grade sand.

The excavation through the broken zone was completed by drilling and blasting in full cross section in rounds of 3 m. A layer of 30–40 cm fibre reinforced sprayed concrete with alkali-free additives, allowing fast build-up of thickness, was planned to be applied as immediate support directly on the frozen ground. A 1–1.2 m thick concrete ring lining was then installed prior to excavation of the next section. The concrete lining was designed to stand a total pressure of 14 bars, estimated as full water pressure plus some soil pressure.

The freezing process was estimated to take 12 weeks, and the subsequent excavation and support 15 weeks.

The tunnel was completed on schedule, principally because the zone had been encountered early and access for continued tunnelling under the fjord was gained through the construction of a bypass tunnel.

A2.2.2 Samanalawewa power project, Sri Lanka

A 4.5 m diameter, 5.35 km long headrace tunnel was constructed as part of the Samanalawewa power project in Sri Lanka. Geologically the site consisted of metamorphosed Precambrian rocks. /Hagenhofer and Pittard, 1990/ gave the following review of the project:

At chainage 1,400 of the outfall drive a completely weathered fault zone was hit. 40–47% of the ground in the following 100 m consisted of a grain size smaller than 75 microns. While these finer grained soils were generally impermeable, significant groundwater was present in the vicinity. When water gained access to the ground ahead of the face along frequently occurring natural slickensided fault planes in the soil, it became prone to sudden collapse and required extensive support. Excavation by top heading and bench was used to improve the stability of the face.

The standard design for tunnel support reflected the assumption that consistently good rock would prevail. There was no support class defined to cope with soft ground conditions. The excavation equipment was mobilised for hard rock tunnelling maintaining a full-face excavation procedure. The application of heavy support and a division of the tunnel face into top heading and bench was thus very time consuming.

As soon as extended sections of completely weathered rock had to be excavated, problems arose. Frequent smaller collapses led to continuous modification of the original design, but no adequate soft ground concept was introduced. The result of these modifications was somewhat unsatisfactory, since with such a large amount of support no safety for the working area could be achieved; at the same time, stabilisation of the supported tunnel section proceeded very slowly.

With the amount of soft ground encountered continuing to increase, the contractor consulted an expert to advise on a soft ground tunnelling concept which followed the principles of the NATM. The target of the new design was to find a support system that would be safe, would require less support material and would allow better progress. The idea was to modify the excavation profile to a rounder one, thus making the structure of the shotcrete lining approach the line of compression more closely. The same thickness of shotcrete as before was used to a greater extent. Design calculations were made using the prevailing shear failure mode as the basis.

The work cycle had to change completely to implement the new soft ground tunnelling concept and the machinery had to be adopted to the 'short bench method'. The top heading was excavated two to three rounds in advance of the bench using an excavator boom.

The use of this technique stabilised the face. Regular holes were drilled to relieve water pressure in cases of excessive pore water pressure. Skill in applying shotcrete for stabilising weak and running ground improved quickly so that the most difficult ground conditions could be managed safely with good progress. Fast progress contributed to stability by reducing the exposure time at the excavation surfaces substantially.

A2.2.3 Tuzla tunnel, Turkey

The shallow (typically 15–20 m below surface) Tuzla tunnel in Turkey was constructed through a sequence of shale and limestone that contained a number of fault zones. /Dalgic, 2003/ presented details of the ground conditions, failure modes and solutions. An extract taken from this paper is as follows:

The rock mass encountered in the zones was typically poor to very poor (GSI values ranged from 10–40). Groundwater with a discharge rate of 100 l/min issuing from fault zones in the excavation face caused softening and flowing of the fault clays as well as block fall, rock slide, ravelling and flowing processes in brecciated and blocky fault fills. To counter this a combination of the following measures were taken; jet grouting, a forepole umbrella, drainage, partial face excavation, reduction of the number of excavation steps and face support. The combination of rock bolts and a forepole umbrella were found to be quite successful in zones of blocky, brecciated rock. However, this combination was less effective in zones that were dominated by clays due to the difficulty in achieving adequate anchorage. As a result, such zones were supported by steel sets.

A2.2.4 Bjorøy tunnel, Norway

The Bjorøy tunnel is located on the west coast of Norway outside Bergen and passes under the straight of Vatlestraumen. The maximum depth of the tunnel is 80 m below sea level. Rock conditions in the area consist of Precambrian gneisses and granites together with Caledonian allochtonous units. /Holter et al. 1996/ give an account of the difficulties encountered in driving this tunnel. An extract from their paper is presented below:

The bedrock was thought to be of good to fair rock mass quality. Thus, hard rock techniques for pregrouting, excavation and support were anticipated throughout the tunnel. Difficult conditions were encountered in about 150 m of the tunnel length when a vertical sheet-shaped zone of heavily jointed and altered Jurassic rock was intersected at an angle of 30–35°. Until the excavation of the tunnel, Jurassic formations had not been known to occur within gneisses of the Bergen region.

Soft sand and silt with Jurassic fossils occurred within the Jurassic rocks both as continuous layers and discontinuous lenses. When encountering the zone with probe drilling ahead of the tunnel face at a distance of 8–10 m, several cubic metres of sand and silt were flushed into the tunnel through the drillholes together with water leakages of about 200 litres per minute. Hence the untreated condition of the soil was assumed to behave like running ground.

A full-scale trial test programme in the tunnel was undertaken to provide the necessary basis for selecting the methods of ground treatment and excavation as well as temporary and permanent support. Engineers decided on a combination of conventional grouting in combination with gravity drainage.

Treatment of extremely poor rock

Treatment of zones of "extremely poor rock" faced two major problems. The first of these, drillability, was caused by the rapid change of mechanical strength and density of open or soil filled joints. To combat this the development of the technique of sectional grouting and re-drilling through steel pipes was utilised. The second problem was to achieve desired penetration of grout into the seepage. After preliminary testing it was realised that it was impossible to achieve sufficient treatment of the rock mass in one single grouting round. The final grouting procedures and fan layouts consisted of a two-stage procedure. The first stage (treatment type 1) was aimed at filling of open joints and channels. The second stage (treatment type 2) was aimed at penetrating the permeable fine joints and fractures with rock contact. The grouting philosophy of the two-stage treatment is compiled in Table A2-1.

Table A2-1. Two-stage treatment of extremely poor rock /after Holter et al. 2001/.

Treatment type	Grout type	w/c ratio	Grouting pressure	Stop criterion
1	Rapid hardening microcement	0.4–0.5	5–10 bar over static water pressure	Fixed amount per drillhole metre
2	Rapid hardening ultrafine microcement	0.7–1.0	15–20 bar over static water pressure	Pressure, 25 bar over static water pressure

Treatment of soil

The dividing between soil and rock mass was based on the potential failure mechanism. Hydraulic failure in the soil zones due to the static water pressure was identified as a potential problem. This issue was increased when grouting with a certain pressure took place and zones of soil were exposed at the tunnel face.

Four different types of grouting product were used to stabilise the soil zones. Two rapid hardening cement types of different grainsizes were used to compact the soil, fill voids (replacement) and help combat hydrofracturing. A rapid setting acrylic resin was used to permeate the soil and to handle unfavourable hydrofracturing towards the face together with a fast setting polyurethane grout. Grout was delivered using sectional grouting and re-drilling through steelpipes. Sections were typically 0.3–0.5 m long in the untreated soil zone.

Engineers considered that it was critical for the feasibility of the chosen concept, that the water pressure had to be removed from the ground near the tunnel contour. As a last measure before the commencement of excavation after treatment, a fan of drainage holes was drilled. The drainage hole fan covered the entire volume around the tunnel contour.

Temporary support of the zone consisted of:

- *10 cm of steelfibre reinforced shotcrete covering the entire contour, with the exception of the floor.*
- *Shotcreted ribs of 7 rebars mounted on 2 m radial rockbolts. The distance between each rib was 1.2–1.8 m and the width 0.4–0.6 m. Shotcrete thickness was 25 cm.*
- *Cast-in-place concrete invert with reinforcement in the wall/floor connection.*
- *Support ahead of the face by 6 m long spilling bolts, partly installed through steelpipes.*

In addition, temporary support of the tunnel face was necessary where the sandzone had an unfavourable exposure at the actual face. The support consisted of 6 cm steelfibre reinforced shotcrete, swellex bolts and short drainage holes.

Engineers calculated that the temporary support could have been sufficient as permanent rock support only with a small increase in the thickness of the shotcrete. However, a conservative approach assuming full static water pressure on the support was taken and a 40–70 cm thick permanent concrete lining was installed in the entire contour with a semicircular section.

A2.3 Overstressed fractured rock

/Hoek, 2001/ published a paper dealing specifically with the rock mechanical issues associated with overstressed fractured rock or squeezing ground. He refers to several case histories in which squeezing ground was encountered and describes the different methods utilised to tackle the problems associated with this type of rock failure. An overview of the theory, problems and methods to combat squeezing ground, based on Hoek's paper is presented in the section below. This is then followed by a series of case histories in which problems associated with squeezing ground were encountered.

A2.3.1 Overview

/Sakurai, 1983/ used field observations and measurements to suggest that tunnel strain levels in excess of approximately 1% are associated with the onset of tunnel instability and with difficulties in providing adequate support. However, in some cases tunnels undergoing strains as high as 4% have been found not to exhibit stability problems. /Hoek, 2001/ suggests that the stability of a tunnel face is a critical factor in driving large tunnels through squeezing ground. The following extract is taken from /Hoek, 2001/.

Instability of the face not only creates extremely dangerous conditions for the workmen in the tunnel, but it also has a major impact on the on the subsequent behaviour of the tunnel. Unless this instability is dealt with in an appropriate manner, significant damage may occur in the rock mass surrounding the tunnel due to the formation of cavities from collapse of material at the face or through gaps in the support system. This damage may require time-consuming and expensive treatment once the face has advanced through the fault, or, if left untreated, it may cause problems later during the operating life of the tunnel.

If the fault is anticipated, for example by probe drilling ahead of the face, and a well-designed plan of attack is developed by the tunnel designer and the contractor, the squeezing problems can usually be overcome. The rock mass surrounding the tunnel may be improved by grout injection, placement of grouted pipe forepoles, or reinforcement with grouted fibreglass dowels (installed in the face). While this treatment will be slow and expensive, it is more likely to succeed and to minimize subsequent problems than the more typical approach where no pre-reinforcement is used and where problems are dealt with as they are exposed at the face.

/Hoek, 2001/ divides methods for dealing with squeezing ground into three categories (Figure A2-1). *One of these involves driving small-sized headings in advance of other portions of the face. This method, which tends to be favoured by tunnel designers north of the Alps, relies on the fact that smaller faces require less support and the sequential construction process results in the creation of a very strong shotcrete shell. The alternative approaches, typically used by tunnel designers from south of the Alps, are to drive a tunnel full-face or by top heading and bench excavation, and to rely on reinforcement of the face and the rock mass surrounding the tunnel to stabilize the tunnel.*

All of the approaches illustrated in Figure A2-1 have advantages and disadvantages, and there are no simple rules for deciding which method is better for a particular set of circumstances. For relatively mild squeezing conditions rockbolts and shotcrete are used as the primary elements in all of these support systems. In the case of the multiple heading method, the face is divided into a larger number of headings as squeezing becomes more severe. This ensures that the outer reinforced shotcrete shell is not overstressed at any stage of the excavation process. The stability of the smaller faces is also easier to control.

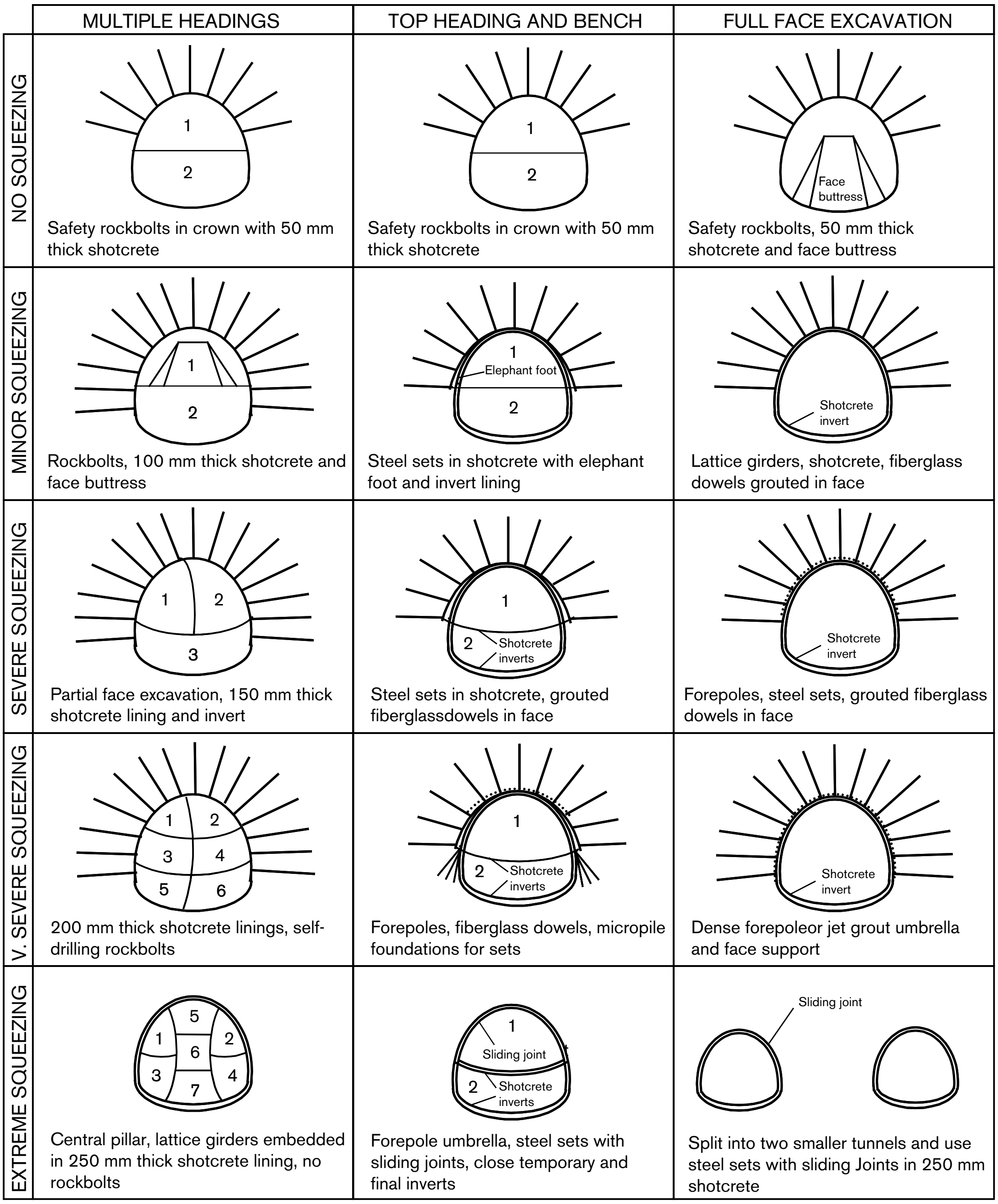

Figure A2-1. *Face excavation and support options for large tunnels /Hoek, 2001/.*

For the top heading and bench and full-excavation operations, heavier and more closely spaced steel sets are added as the severity of the squeezing increases. For very severe squeezing conditions, grouted fibreglass dowells are added for face reinforcement and forepoles or similar reinforcing elements are used to prereinforce the rock mass ahead of the advancing face.

While this prereinforcing is very effective in protecting the rock core ahead of the face, it can become a liability once it is exposed at the tunnel face, where the exposed ends often need to be supported by steel sets. It is particularly important that foundation failure of the bases of the steel sets is prevented by the provision of some form of footing or anchoring system. A frequent design error is the use of excessively large forepoles that, while providing good support for the rock mass ahead of the face, tend to overload the steel sets behind the face.

Eventually a point is reached where it is difficult to provide support of sufficient capacity, particularly if extremely severe squeezing is associated with very poor quality rock masses in which rockbolts are ineffective. In such cases it may be necessary to allow the support to yield in a controlled manner so that its capacity is only mobilized after significant displacement.

In very poor ground it is difficult to keep drillholes open and it may be necessary to use self-drilling rather than conventional rockbolts. These are rockbolts fitted with disposable drill bits that are left in place at the bottoms of the holes. In extremely poor ground, particularly where clay minerals are present, self-drilling rockbolts may be ineffective because of failure of the bond between the grout and the surrounding rock.

The final choice of the method to be used for a specific situation depends upon the complex interaction of a number of factors. In addition to safety, cost and schedule considerations, these factors also include the relevant experience of the contractor, designer, and consultants engaged to assist in the project. The successful implementation of the methods illustrated in Figure A2-1 depends more on experience-based judgement than on theoretical calculations. In particular, the experience of and the authority given to directing the work at the face is crucial, since there is seldom time for lengthy academic discussions when dealing with unstable tunnel face problems. Wherever possible this individual should be an engineer, because it is not only experience but also an understanding of the mechanics of rock-support interaction that will dictate the choice of the most appropriate course of action.

A2.3.2 Summary of case histories of squeezing tunnels

A table containing information on a variety of tunnel projects that encountered squeezing ground is presented below (Table A2-2). More comprehensive descriptions of several case histories are detailed in the previous sections.

Table A2-2. Case histories of squeezing tunnels /Hoek, 2001/.

Tunnel name, location and rock type	Depth (H) – m	Rock mass strength (σ_{cm}) – MPa	* σ_{cm} /γH	Tunnel span – m	Closure – m	Strain ε %	Comments
1. Yacambu-Quibor, Venezuala (graphitic phyllite)	600	1.0	0.06	5.5	5	> 30	Extreme squeezing, stability controlled by yielding steel sets /Sánchez Fernándes and Terán Benítez, 1994/
2. Nathpa Jhakri headrace tunnel, India (fault zone)	300	0.6	0.25	10	2	20	Severe squeezing, stability controlled by forepole umbrella /Hoek, 1999/
3. Maan headrace tunnel, Taiwan (sandstone/shale)	200	1.6	0.33	6.5	0.1	1.5	Mild squeezing with local shotcrete damage /Chern et al. 1998/
4. Maan project, Adit A, Taiwan (sandstone/shale)	200	0.7	0.14	6	0.22	3.7	Large squeezing with support damage /Chern et al. 1998/
5. New Tienlun headrace tunnel, Taiwan (fault zone)	400	0.7	0.07	6.5	0.9	14	Severe squeezing with local tunnel collapse /Chern et al. 1998/
6. Mucha tunnel, Taiwan (sandstone/shale)	110	1.4	0.49	16	0.16	1	Stable tunnel /Chern et al. 1998/
7. Mucha tunnel, Taiwan (fault zone)	120	0.28	0.09	16	2.4	15	Severe squeezing with local tunnel collapse /Chern et al. 1998/
8. Pengshan tunnel, Taiwan (sandstone/shale)	140	1.9	0.55	12	0.01	0.11	Stable tunnel /Chern et al. 1998/
9. Maneri-Uttarkashi power tunnel, India (metabasics)	800	2**	0.1	4.75	0.43	9	Severe squeezing, damage to sets and concrete lining /Goel et al. 1995/
10. Chibro-Khodri tunnel, India (crushed red shale)	280	0.7**	0.1	3	0.01	2.8	Moderate squeezing, stabilized by circular steel sets /Singh et al. 1992/
11. Giri-Bata tunnel, India (slates)	380	0.8**	0.08	4.2	0.3	7.6	Large squeezing with deformation of steel sets /Singh et al. 1992/
12. Giri-Bate tunnel, India (phyllites)	240	0.7**	0.1	4.2	0.38	9	Severe squeezing with buckling of steel sets /Singh et al. 1992/
13. Loktak tunnel, India (shale)	300	0.7**	0.1	4.8	0.34	7	Large squeezing, supported by rock bolts, shotcrete and sets /Singh et al. 1992/
14. Maneri Bhali Stage I, India (fractured quartzite)	350	1**	0.1	4.8	0.38	7.9	Severe squeezing with buckling of steel sets /Singh et al. 1992/
15. Maneri Bhali Stage II, India (sheared metabasics)	410	3**	0.28	7	0.2	3	Mild squeezing /Singh et al. 1992/
14. Maneri Bhali Stage III, India (Metabasic rocks)	480	3**	0.24	2.5	0.06	2.5	Mild squeezing /Singh et al. 1992/

* γH: γ = unit weight of rock, H = depth of tunnel; σ_{cm} = rock mass strength based on /Hoek, 2001/.

** Estimated from the descriptions provided by Hoek and from his personal experience of Indian rock types.

A2.3.3 Yacambú-Quibor tunnel, Venezuela

The Yacambú-Quibor tunnel is a 5.5 m span, 25 km long water transmission tunnel being excavated through the Andes near the city of Barquisemto in Venezuela and is due to be completed in 2006. The maximum cover in the tunnel is 1,270 m and a significant proportion of the rock through which it is mined is graphitic phyllite. /Hoek, 2001/ gives the following overview of the problems associated with this tunnel:

Construction of the tunnel, regarded by many as the most difficult tunnel in the world, commenced in 1975. In 1979 a tunnel boring machine was trapped by squeezing rock during a stoppage in the drive. The machine could not be restarted and the squeezing rock gradually filled all the cavities in the machine structure. The remains of the machine were removed several years later. Complete closure of the tunnel occurred in several locations and a technique was eventually developed to control the stability of the tunnel by installing steel sets with sliding joints that locked after the tunnel had converged about 0.3 m (equivalent to a strain of approximately 6%) after installation of the sets. These sets were fully embedded in shotcrete except for 1 m wide gaps over the sliding joints. After the tunnel face had advanced 5 to 10 m, the joints had closed and the sets began to accept load. The gaps were filled with shotcrete to complete the lining. Convergence measurements have shown that these sections are stable, and the long-term behaviour of this support has been excellent /Sánchez Fernándes and Terán Benítez, 1994/.

A2.3.4 Nathpa Jhakri Hydropower Project, India

According to /Hoek, 2001/, severe deformations occurred when tunnelling through a wide fault zone in a 10 m diameter hydropower headrace tunnel under a cover of 300 m at the Nathpa Jhakri Hydropower Project in India. The deformed section was remined under the protection of a forepole umbrella. The remainder of the fault zone was successfully traversed using this method.

A2.3.5 Mucha highway tunnel, Taiwan

The 16 m span Mucha highway tunnel underwent inward displacements of the roof and sidewalls by approximately 1.2 m (equivalent to a strain of about 15%) whilst passing through a fault zone /Chern et al. 1998/. The reduction of the cross section of the tunnel meant that remining of the original tunnel was necessary. This involved heavy support using long tensioned grouted cables that were used to support the failed rock mass while it was mined section by section. The remined section was stabilized by installing additional tensioned grouted cables, and the final concrete lining was placed as soon as possible after completion of the remedial work.

A2.3.6 Inntal tunnel, Austria

The 12 km long Inntal railway tunnel in Innsbruck, Austria, was designed to relieve the Inn Valley's road system of freight traffic between Munich and Verona. The tunnel was driven through glacial deposits, dolomite, quartz phyllite and gneiss. Drill and blast methods were used according to the principles of the New Austrian Tunnelling Method (NATM). Steel arches were placed at 1 m intervals in the tunnel, and steel mesh and shotcrete combined with grouted 6 m–8 m long anchors were used along the entire length /Purrer et al. 1993/. Large radial deformations of up to 220 mm occurred in the top heading when the excavation works encountered a fault zone and the shotcrete lining was significantly damaged. In order to permit the shotcrete to accommodate the deformation without suffering any damage, four longitudinal deformation slots, generally 350–450 mm wide, were constructed in the shotcrete lining (Figure A2-2).

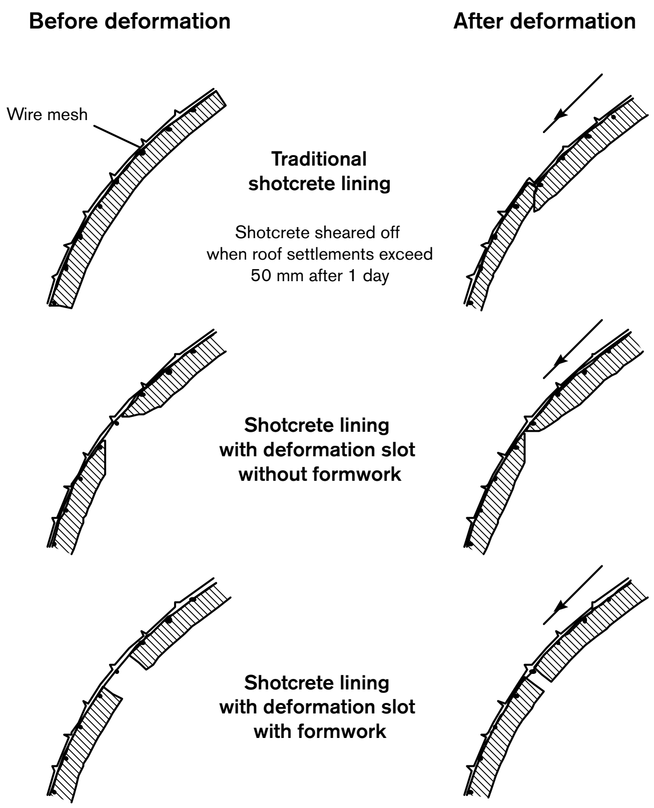

Figure A2-2. *Damage to the shotcrete lining in the Inntal tunnel and development of deformation slots /after Purrer, 1990/.*

A2.4 Swelling

/Selmer-Olsen and Palmström, 1989/ published a review of the processes associated with swelling minerals with respect to tunnelling. In addition, they site several projects in which swelling clays have caused tunnel instability. A review of the more pertinent parts of this paper is presented below:

The following minerals have the capacity to swell and cause stability problems in tunnels:

1. Smectite (montmorillonite, vermiculite etc);
2. Anhydrite;
3. Some pyrrhotites in calcareous shales.

The first group is the most common and is often associated with weakness zones or gouges.

About 75% of the cost of extra reinforcement in Norwegian tunnels after they have been put into operation has been associated with swelling clay. The problems that have arisen have often been caused by the lack of experience on the part of the people involved concerning the swelling mechanism, combined with an underestimation of the capacity for swelling clays to exert high pressures on tunnel linings.

Swelling clays associated with weakness zones occur in two ways:

1. As fillings strictly associated with joints, veins, fractures or faults;
2. As rock-forming minerals in altered rocks.

Usually rock powder and fragments are present together with the swelling minerals; also other secondary minerals such as calcite, quartz, chlorite, talc, zeolites, kaolinite and hydromica/illite may be associated with the swelling clay minerals in the fillings. The type of cation present affects the degree of swelling to a great extent. Na, for example, will cause a higher degree of swelling, than Ca.

The most important factors affecting the degree of swelling in a zone are:

- The amount and type of swelling minerals;
- The amount and type of mobile cations;
- The degree of consolidation of the material in the zone;
- The access to water;
- The degree of unloading after excavation.

In addition to the factors listed above, the amounts, types and shear properties of the other fine-grained, loose materials in the zone will influence the swelling properties and the behaviour of the zone. Calcite and other minerals that can be dissolved, and therefore, space for the softened filling to be washed out, are important in this connection /Selmer-Olsen and Palmström, 1989/.

A2.4.1 Rafnes water supply tunnel, Norway

/Selmer-Olsen and Palmström, 1989/ discuss 4 case histories in which swelling clays have caused sections of tunnels to collapse, both after short and long time periods (i.e. from a matter of weeks to several years after construction). Of most interest to this project is the example from the Rafnes water supply tunnel in Norway.

The Rafnes water supply tunnel, which is partly located below the sea, was excavated in Precambrian gneisses and amphibolites, which are intersected by several zones up to 5 m thick containing clay. In addition, the rocks on both sides of the zones were often altered, partly to swelling clay minerals. Because the zones were dry, only a few of them caused minor stability problems during tunnel excavation, but they were all given initial support by shotcrete. Later, additional shotcrete, often reinforced, was applied as the final rock support.

A few months after the tunnel had been filled with water, but before going into service, it was blocked by several collapses. During inspection of the emptied tunnel it was found that up to 30 cm-thick reinforced shotcrete had been destroyed in about 30 locations and that larger collapses had blocked the tunnel in four places. The reason for this was the occurrence of swelling clay and the fact that shotcrete had been applied shortly after excavation. The rapid spraying of shotcrete after excavation did not allow the clay to swell, resulting in the build-up of high swelling pressures.

A2.4.2 Ellingsøy road tunnel, Norway

Problems associated with swelling clay are well documented in Norway. /Nilsen, 1990/ discuss one such problem encountered in the Ellingsøy tunnel. An extract from this paper is presented below:

In Norway, subsea tunnels beneath fjords often encounter weakness zones, which may have widths of 20–30 metres and in most cases, consist of crushed and weathered rock. The gouge material is often of a swelling type (smectite) and swelling pressures up to 2 MPa have been experienced.

/Nilsen, 1990/ highlights that particularly bad conditions were encountered in the Ellingsøy road tunnel, in which a fault zone containing swelling clay and waterbearing fissures caused a cave-in reaching a maximum height of 8–10 m above the crown before it finally stopped after 24 hours. A concrete plug of approximately 700 m³ was constructed in order to ensure safe tunnelling through the zone /Olsen and Blindheim, 1989/.

In conventional tunnels under land in Norway, a few cases of slides reaching several tens of metres above the tunnel crown have been experienced. Most such incidents have been caused by major faults containing swelling clay. Analytical studies have been carried out to calculate the maximum theoretical propagation of a slide above the tunnel roof and the method is presented in /Nilsen, 1990/.

A2.5 Investigation methods and practical issues for tunnelling through waterbearing fault zones

A2.5.1 Underground petroleum storage cavern – Linköping, Sweden

A large underground petroleum storage cavern was constructed in coarse-grained augen-gneiss in Linköping, Sweden. /Jansson, 1978/ discusses the major issues associated with excavation of the cavern. The following is an extract from Jansson's article:

The roof gallery began without any trouble but later on very broken, weak and highly water-bearing rock was encountered. The greatest single water leak was 20 m³/h. It was decided to drive the roof gallery with a pilot tunnel, followed by two side stopes. The rock both in front of the pilot tunnel face and in the side stopes was pre-grouted from the pilot tunnel. The areas where water leakage occurred even after blasting were clad (covered with) in shotcrete after which the rock was post-grouted.

Pre-grouting for the benches was done through vertical holes drilled from the roof gallery level down into the body of rock, which would later form the cavern walls. Despite this there were quite a lot of leaks after blasting. These were remedied by using shotcrete and post-grouting. Furthermore attempts were made to reduce in-flow through the cavern floor by making a shielding curtain of vertical grout holes down into the rock along the cavern walls.

Very little of the rock outcropped at the surface and the investigation was largely based on a couple of seismic profiles that showed the rock was good, with high wave velocities. No core drilling was done.

While building the caverns it was shown that an upper, horizontally lying slab of rock was reasonably homogeneous, where as the rock underneath it was very cracked. Presumably nearness to a large regional fault or tectonic origin played a large part in this respect. A more precise pre-investigation, which would have included core sampling, would most probably have indicated this very cracked rock carrying a large quantity of water.

A2.5.2 Vexin water tunnels, France

A series of tunnels and shafts situated up to 120 m below the water table were constructed at Vexin, France. Roadheaders were used to excavate the white chalk. The following extract on the Vexin water tunnels is taken from /Jansson, 1978/.

The most severe leakage was encountered when the cutter head cut into a karst-like chimney of about 40 cm diameter and a heavy inflow of about 470 m³/hour occurred. As the installed pump capacity was only 200 m³/h flooding of the worksite was unavoidable. The pump *installation was quickly increased to 1,000 m³/hour and after 10 days the tunnel had been pumped dry. Two metre-thick walls of concrete blocks were made in front of the tunnel face, the water was channelled through pipes and the 4 m long space between the walls was filled with a mixture of cement, bentonite, sand and water to form a plug. Then the water flow through the pipes was stopped and the space between this plug and the tunnel face was filled with grout. Grout holes were drilled from the tunnel in front of the concrete plug at an angle into the rock around the tunnel and these were then grouted. When check holes around the zone were dry the concrete plug was removed and the tunnel continued. A total of 1,100 m of grouting holes and check holes were drilled and 823 tons of cement grout were used. The total stoppage time to remedy the inflow was 2.5 months.*

A sequence of test shafts and tunnels were constructed prior to driving the main tunnel and in these tunnels a variety of testing was conducted. /Jansson, 1978/ suggests that on the basis of these measurements the Owner had calculated a theoretical tunnel profile, which in practice was shown to be well suited to the strength conditions in the weak rock material. However, /Jansson, 1978/ also states "in the authors opinion, the Owner had not managed to predict the often parallel and nearly vertical heavily water-bearing regional joint sets which separated zones of relatively homogenous and impervious rock and which during the construction of the plant caused great problems with water. The trial tunnel at level –125 m appeared, unfortunately to have been completely in a zone of homogeneous and impervious rock."

/Jansson, 1978/ proposed that the following points should be considered when driving tunnels in highly water-bearing rock:

- The pre-investigation must be so comprehensive and accurate that it can render a clear picture of the underground areas in question. If the investigation results show that difficulties can be expected then alternative positions should be looked at.
- Systematic soundings should be carried out from the start. If the drilling work for sounding can be smoothly fitted into the driving rhythm stoppages should be considerably reduced.
- Grouting facilities must be held in readiness all the time. Mobile equipment, which can follow the advance of the face, is preferable to long pipe systems and central mixing and pump stations. Pre-grouting is the best method and ought to be carried out as much as possible.
- Pumping installations for keeping the tunnels dry during the construction period must be dimensioned from the start to cope with normal leakage as well as the largest, sudden inflow that is judged to be possible.

A2.5.3 The Orange Fish Tunnel, South Africa

The Orange Fish Tunnel is a 5.3 m diameter tunnel stretching 82.5 km. The tunnel was constructed mainly in sedimentary rocks, consisting of more or less horizontally bedded sandstones, siltstones and mudstones. Also present in the sedimentary rocks was a network of dolerite intrusions varying from horizontal sills to vertical dykes. The following extract on the Orange Fish tunnel is taken from /Keyser and Krige, 1978/.

Groundwater was generally encountered at shallow depths above the tunnel. However, abnormally high-yielding, steep-dipping contact zones of dolerite dykes were encountered along limited lengths of the tunnel. The peak inrush of water was calculated to be some 20,000 m³/hour, the tunnel being some 119 m below ground level. The dewatering pumps *could not cope with the water and some 1,750 m of tunnel was flooded. It was necessary to determine the extent of the high-yielding fracture zone and for this purpose an extensive programme of exploratory drilling was done from the surface. Once the work was completed a comprehensive system of grouting, also from the surface, was undertaken to stem the inrush of water. Grouting was done from vertical holes in a radial pattern around the problem area.*

On completion of the grouting programme, the affected section of the tunnel was dewatered and the tunnelling resumed. As a safety measure long pilot boreholes were drilled ahead of the advancing face.

A2.5.4 Jonkershoek Tunnel System, South Africa

The Jonkershoek tunnel system comprises 31.3 km of 3.5–4.3 m diameter water tunnels in the Western Cape. The tunnels pass under folded and severely faulted mountain ranges with depths of cover in excess of 1,000 m. Reasonably sound sandstones and granites were encountered together with completely decomposed and highly jointed water filled fault zones even at the maximum depths of cover. The following extract on the Jonkershoek tunnel is taken from /Keyser and Krige, 1978/.

At the heading of the Franschoek mountain section, a few badly decomposed fault zones were encountered. The first fault zone was encountered when the face had been advanced some 3,000 m and the cover above the tunnel was 1,000 m. This fault zone occurred at the contact between sound granite and sandstone. The tunnel collapsed and a serious inflow of water and mud from the decomposed granite in the fault zone was experienced.

After unsuccessful attempts to stabilize the 50 m long fault with high grouting pressure, the engineers decided to adopt a top heading and bench method of excavation. The heading was driven with the help of spilling knives and Bernold Plates. Normal tunnelling was resumed after 22 months work through 50 m of clay.

A second serious fault was encountered after a further 1,000 m of tunnelling. It was however, possible to stabilize this zone with high pressure grouting.

A third fault zone was encountered in which conditions were so bad that very special measures had to be taken to ensure that tunnelling would proceed. Six faults were identified during exploratory drilling. A grading analysis of the material in the fault zone showed that it contained up to 20% with a grain size smaller than 150 micron. Laboratory tests also indicated that it could not be drained due to the fine grain size.

The stability of the fault zone material was examined in a test adit. When a part of the fault was exposed it was found that within 15 minutes of exposure the fault material was slowly moving into the excavation. This experiment proved that tunnelling through the fault zone would require very special methods and it was decided that the only sure method would be to freeze the fault zone.

In addition to ground freezing, the tunnel's alignment was also altered in order to intersect the zone perpendicular and hence minimise the width of the zone to be tunnelled through.

A2.6 Summary

The case histories presented in this appendix provide an insight into tunnels that have encountered similar problems/ground conditions as those concerned in this study. It is evident that the method selected for tackling these conditions is case specific. What is clear is that characterization of the zone and preparation of a "plan of attack" prior to exposing the zone in the tunnel face is critical. Lack of sufficient site investigation is sited as a cause for the problems encountered at the Linköping petroleum storage project and for the Vexin tunnel project. This could also apply to several of the other case histories discussed in this chapter. The use of advance probe drilling to locate and characterize weakness zones is a particularly vital part of the investigation process and was used at Oslofjord, Bjorøy and at the Orange Fish tunnel. In addition, where waterbearing fracture sets are suspected, it is important to ascertain their orientation during the site investigation in order to avoid driving the tunnel parallel to a major waterbearing fracture system, as was the case in the Vexin tunnel.

Practical issues, such as ensuring that sufficient pump capacity for the largest sudden inflow is available, are important to maintain a safe working environment, and to ensure that the tunnelling process is as effective as possible. The working sites at both the Vexin tunnel and the Orange Fish tunnel were flooded due to insufficient pump capacity.

Modification of the tunnel route to minimise the length of section of the fault zone (e.g. Jonkershoek tunnel system), driving of bypass tunnels (e.g. Oslofjord) and adits/niches (e.g. Linköping petroleum storage), to work multiple faces can speed up and assist the passage of weakness zones.

There are a multitude of combinations to combat squeezing ground and sections of poor rock. Reduction of the size of the working face (multiple headings/top heading and bench), have been employed on many tunnel projects, such as Samanalawewa, Tuzla and the Linköping petroleum storage in order to improve control of tunnel stability. Use of a forepole umbrella to provide support to the ground ahead of the tunnel face was employed at Tuzla, Bjorøy, Nathpa-Jhakri and at the Jonkershoek tunnel system, however, /Hoek, 2001/ warns about the problems associated with using forepoles which are too long and overstress the tunnel support at the face (commonly steel sets). Provision for the increase in load at the base of steel sets can be made to avoid foundation failure. Support of the tunnel face in poor ground in the form of shotcrete and dowels (Bjorøy) can be used to control face stability. Alterations of the tunnel's form (Samanalawewa) and reinforcement of the invert (Bjorøy) are methods that enable the tunnel to accommodate high stress environments. Self-drilling rockbolts can be used in poor rock where drilling is difficult. Steel sets are commonly used (e.g. Tuzla) in conditions where rockbolts cannot gain sufficient anchorage. Tensioned, grouted cables offer an alternative to rockbolts where heavier support is required or where the distance to a firm anchorage is larger than normal. Grouted cables were successfully used in the Mucha Highway tunnel and the Inntal tunnel. In situations where extremely large deformations are anticipated, the support may need to be designed to accommodate a degree of deformation before it becomes active. Such examples include the Yacambú-Quibor tunnel (steel sets) and the Inntal railway tunnel (shotcrete).

Drainage holes have been used in several projects to reduce the local hydraulic gradient and improve drilling conditions. Examples in which drainage methods were employed include Samanalawewa, Vexin, Bjorøy and Tuzla.

Fault zones may contain swelling minerals that can exert large pressures on tunnel linings if no provision for them is made. If shotcrete is applied too rapidly, swelling minerals are unable to expand and hence apply pressure to the lining, which can lead to failure at a later date (e.g. Rafnes).

Pre-grouting has been commonly used to combat weakness zones. Not only does it reduce the water inflow, but it also enhances the mechanical properties of the rock. Projects in which pre-grouting was used include Tuzla, Vexin and Linköping. A preliminary phase of pre-grouting, prior to ground freezing was also used at Oslofjord. The Bjorøy tunnel project used a 2 stage grouting process in poor rock and 4 different grout types in soil sections. Difficulties associated with the drilling process were tackled by using sectional grouting with steel pipes.

Two projects in which grouting failed due to high water pressures and bad ground conditions were a section of the Jonkershoek tunnel system and the Oslofjord crossing. As a result, ground freezing was employed in both projects to facilitate the passage of these zones.

Appendix 3

A3 Stochastic analysis of block/wedge stability

A3.1 General

The mechanical behaviour of hard jointed rock mass is highly affected or even determined by its blocky structure. The blocky structure is defined by discontinuities, orientation, spacing and extension. The jointed rock mass is a stochastic medium, thus discontinuity properties and mechanical properties are actually random variables. It is desirable to describe mechanical behaviour of jointed rock mass with knowledge of discontinuities locations and geometry around the tunnel. Comprehensive and precise description of rock mass discontinuities is impossible because of the nature of the rock mass. Nevertheless, methods for a statistical description of the discontinuities network are available from geological survey data analysis.

From rock engineering's point of view, it is desirable to predict the volumes of potentially unstable rock blocks in the preliminary design phase, so that types of rock supports can be determined accordingly.

The solution proposed by MSB (Method for Simulation of Blocks) method is to empirically evaluate these characteristics, by running series of numerical experiments. The general idea is to build a 3D statistical model of the jointed rock mass statistically equivalent to the natural jointed rock mass. MSB is a concept and computer code for modelling hard blocky rock mass and deriving stability parameters from statistical description of joint system and geological survey data. It is applicable for tunnel and tunnel support design including rock bolt system design. MSB is a stochastic approach, coherent with statistical description of discontinuities. It is a probabilistic extension of the Block Theory. It is replicating unstable block systems around a tunnel in the schema of Monte-Carlo simulation and predicts stability parameters in the means of statistics. The method and code was originally formulated by /Jakubowski, 1994, 1995/ and then modified and improved.

A system of discontinuities is generated around a tunnel excavation according to an assumed statistical model, then the removable blocks are identified and the static limit equilibrium equations are applied to the removable blocks. These steps are repeatedly performed in the general scheme of the Monte-Carlo statistical simulation. Multiple simulation produces a sample which is a base for calculating statistical characteristics of the mechanical behaviour of a blocky rock mass.

The method of modelling rigid blocks system around a tunnel is based on:

- Statistical Monte-Carlo simulation scheme,
- The key block concept and the limit equilibrium stability analysis,
- The original algorithm of removable block identification,
- Unstable area probability map concept and other specific output statistics.

A3.2 Jointed rock mass modelling

The essential problem is virtual generation and numerical investigation of the rock mass blocky structure statistically equivalent to the true jointed rock. It is assumed that the statistically homogeneous rock mass region is intersected by planar discontinuity surfaces of arbitrary orientations. The discontinuity planes separate rigid blocks, indivisible beyond the planes of discontinuity. The tunnel support should ensure the stability of blocks not stabilised by gravity, other blocks or friction. The generation of discontinuity planes system around tunnel is run according to the following statistical discontinuity model:

- Orientation of each set of joints follows the best fitting distribution function on a sphere or hemisphere,
- Spacing of each set of joints follows the best fitting distribution function,
- Orientation and spacing are statistically independent random variables,
- Extension of discontinuity planes creating blocks is restrained by a probability size filter.

If two planes or a plane and edge of a specific block are parallel or close to be parallel, it is very possible for the block to be stabilised by joints, waviness or roughness. The narrower, thinner and longer the block is the easier it is to be broken apart and then stabilised. A “shape” filter is applied to all the removable blocks. This is a procedure, rejecting some blocks from the set of removable blocks (assuming, they are non-removable).

Input parameters values are based on the statistical analysis of geological survey data collected by outcrop description, drillcore and drillhole description or photogrammetry. Depending on available geological data and the technique applied to describe joints system effective joints system parameters are estimated to provide the best possible projection of real joints density, traces length and blocks size around a tunnel.

MSB simulates the effect of rock bolt reinforcement for the stability of excavation. Rock bolts are considered as structural elements of blocks system. The input parameters for the MSB analysis incorporating rockbolts reinforcement include: pattern (spacing), length and ultimate rock bolt axial capacity. The stability of identified removable blocks is analysed by the limit equilibrium method.

The discontinuities system generation, removable blocks analysis and stability analysis are repeatedly undertaken according to the general Monte-Carlo simulation scheme. The number of simulations (computation cycles) depends on required estimation closeness. The following factors are controlled and checked during the multiply simulation:

- Distribution of the generated discontinuity planes orientation and spacing;
- Edge effect (influence of the model boundary on the blocks distribution along the tunnel axis);
- Uniformity of the block distribution along the length of the modelled tunnel segment;
- Closeness of estimation.

A3.2.1 General scheme

The general scheme of the modelling method is presented in the figure below. Geological survey data is first statistically analysed, then discontinuity planes generated, blocks identified, limit equilibrium stability analysed (within the multiply simulation scheme) until the method finally provides various estimations of stability factors and support forces preventing failure. For specific discontinuities network and blocks, any defined parameter of blocks, load and stability (coherent with the assumed physical model) can be computed and analysed and finally summarised in the form of various statistics, probability distributions and estimations of failure risk.

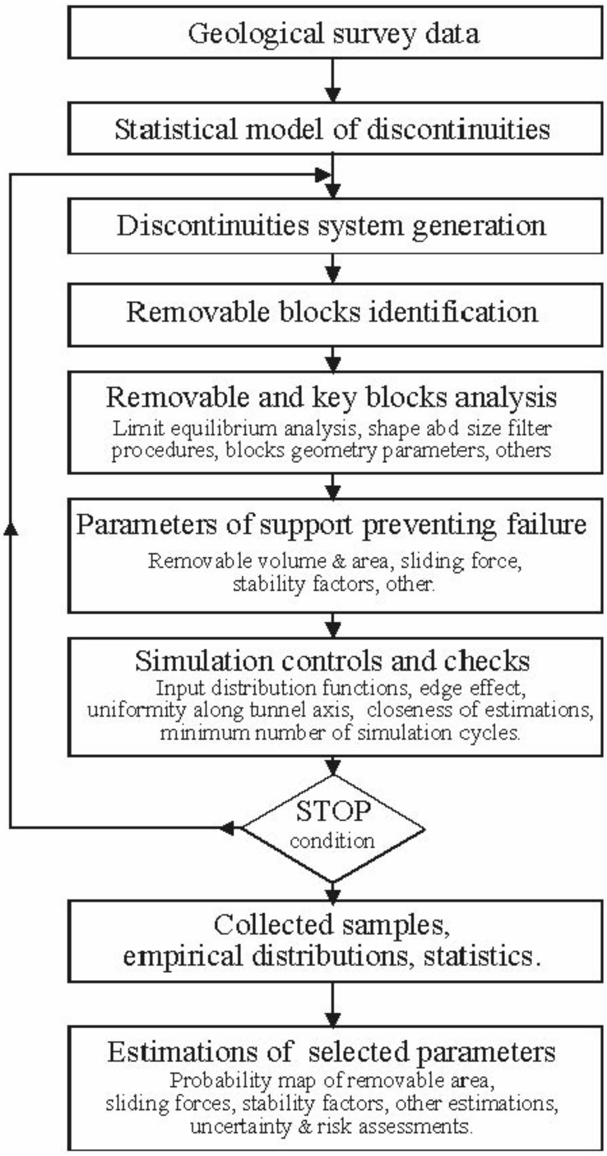

Figure A3-1. *General scheme of MSB simulation cycle.*

The presented method is a modelling tool allowing estimating parameters appropriate for tunnel and tunnel support design in the certain geological and technical circumstances. Both quantitative and qualitative output and risk estimations can be used for tunnel design purposes. The shape and size probability filters concept can be used for model calibration.

A3.3 Unstable area probability map

Unstable area probability map is one of output statistics and presentation of MSB simulation results. The concept of unstable area probability maps was originally proposed by /Jakubowski, 1994, 1995/. For the certain system of planes and blocks geometrical and limit equilibrium analysis is performed and unstable blocks are identified. The network of points located around the tunnel are then checked to see if a specific point belongs to any unstable block or not. The set of points belonging to any unstable blocks create unstable volumes. Projected on the cross-section plane it gives an unstable area map. The same procedure repeated for many sets of data (following statistical distributions based on field measurements) gives probability maps of unstable areas. It describes the geometrical probability that specific point in the vicinity of the tunnel belongs to any unstable block. It also describes the stability status of points in the vicinity of tunnel cross section in relation to their locations. It presents a probabilistic measure of the instability of the rock mass in the vicinity of the tunnel. Presented are not only the value of stability measure itself, but also the extension of unstable areas within tunnel walls and specific distribution of unstable area around tunnel cross-sections.

A3.4 Analyses based on data of NE-1

A3.4.1 Geological data and assumptions

Among several sets of discontinuities mapped in the region of NE-1 in the Äspö tunnel, three major fracture sets were selected and considered for the simulations: J1, JW1 and JW1 (see Appendix 1). JW1 and JW1 are water-bearing fractures and are deemed as significant structures for stability aspects, while J1 is one of the most frequent fracture sets (see Appendix 1). A tunnel length of 35 m is simulated.

The available data on discontinuity orientation and spacing were not complete and coherent and their interpretation introduced some significant uncertainties. This is why additional assumptions about discontinuities extensions had to be adopted. The available geological documentation did not provide reliable measure of discontinuity trace lengths or discontinuity extensions and did not allow estimating discontinuity intensity in any strict manner. The input joint extension parameter (corresponds roughly to an average trace length) was set up to 9.0 m for all the three discontinuity sets. The available data on discontinuities spacing was also unconvincing, since it is given as spacing measured perpendicular to joints traces along free faces. Such measurement results depend on the orientation of free face, and are different if measured on the walls or on the floor. With no information on where the specific joint spacing was measured[1], it was not possible to recalculate available data into unified and comparable value of spacing[2]. In order to handle this problem it was assumed that joint spacing was measured on the walls, for all joints. Moreover, it was assumed that the most commonly valid functions are applicable for the spacing distribution and orientation distribution: spacing is exponentially distributed and joints orientation follows Fisher distribution (Table A3-1) where the dispersion parameter is a measure of the variance in the orientation data within each fracture set.

Table A3-1. Joint set description for the basic set of data.

Joint set	Mean spacing (*)	Spacing distribution function	Joint extension parameter	Friction angle	Mean orientation dip/dip dir	Orientation probability distribution function	Fisher pdf dispersion parameter
	(m)		(m)	(deg)	(deg/deg)		
J1	0.57	exponential	9.0	30	90 / 14	Fisher	50
JW1	0.42	exponential	9.0	30	35 / 320	Fisher	50
JW2	0.87	exponential	9.0	30	45 / 71	Fisher	50

(*) Mean spacing given above is the spacing perpendicular to the mean plane of joint set (measured along the mean joint set orientation axis). It was recalculated from the given spacing data.

[1] Important is where was the spacing measured, on the walls or floor/roof, what is the orientation of the free face where specific traces were located and measured.

[2] Spacing measured along mean joint set orientation axis.

For the purpose of the prediction of unstable blocks, the following cases are simulated:

1. Friction angle = 0° for the all fractures, no support;
2. Friction angle = 30° for the all fractures, no support;
3. Friction angle = 45° for the all fractures, no support;
4. Friction angle = 30°, supported with bolts of length 4 m; spacing 2 m and load bearing capacity 150 kN.

Cohesion of the fractures are set to 0 MPa for all the cases.

General data for simulations:

Tunnel axis azimuth: 352°.

Tunnel axis inclination (dipping north): 8°.

Tunnel width: 7 metres.

Tunnel maximum height: 8.5 metres.

Tunnel section length: 35 metres.

Number of joint sets: 3.

Rockbolt spacing (option 4 only): 2 metres.

Rockbolt length (option 4 only): 4 metres.

Rockbolt capacity (option 4 only): 150 kN.

A3.4.2 Results of the simulations

A summary of the simulation results is given in Table A3-2, while Table A3-3 gives an estimated frequency for the volume of unstable blocks. Figure A3-2 shows the histograms of estimated block volumes and Figure A3-3 shows "probability maps" of unstable areas around the tunnel.

Table A3-2. Summary statistics of prediction of unstable block volume.

	Mean no of blocks per 35 m tunnel section	Block volume statistics						
		Mean (m^3)	Median (m^3)	Minimum (m^3)	Maximum (m^3)	Lower quartile (m^3)	Upper quartile (m^3)	Std dev (m^3)
Case 1	649	0.76	0.12	0.00	179.78	0.010	0.61	2.16
Case 2	392	0.69	0.10	0.00	46.63	0.011	0.55	1.83
Case 3	206	0.55	0.08	0.00	47.68	0.008	0.44	1.51
Case 4	284	0.19	0.04	0.00	21.00	0.004	0.18	0.43

Table A3-3. Unstable block volume frequency tables.

Case 1 Volume category (m^3)	Mean number of blocks per 35 m tunnel section	Percent	Case 2 Volume category (m^3)	Mean number of blocks per 35 m tunnel section	Percent
Vol < 0.1	307.7	47.43	Vol < 0.1	194.2	49.60
0.1 < = Vol < 1.0	225.9	34.82	0.1 <= Vol < 1.0	132.9	33.93
1.0 < = Vol < 2.0	53.2	8.20	1.0 <= Vol < 2.0	30.0	7.66
2.0 < = Vol < 5.0	43.0	6.63	2.0 <= Vol < 5.0	23.7	6.06
5.0 < = Vol < 10.0	13.4	2.06	5.0 <= Vol < 10.0	7.87	2.01
10.0 <= Vol < 20.0	4.49	0.69	10.0 <= Vol < 20.0	2.46	0.63
20.0 <= Vol	1.16	0.18	20.0 <= Vol	0.43	0.11
Sum	648.9	100.0	Sum	391.6	100.0
Case 3 Volume category (m^3)	**Mean number of blocks per 35 m tunnel section**	**Percent**	**Case 4 Volume category (m^3)**	**Mean number of blocks per 35 m tunnel section**	**Percent**
Vol < 0.1	109.4	53.02	Vol < 0.1	184.4	65.02
0.1 < = Vol < 1.0	68.3	33.12	0.1 < = Vol < 1.0	87.7	30.94
1.0 < = Vol < 2.0	14.2	6.88	1.0 < = Vol < 2.0	8.82	3.11
2.0 < = Vol < 5.0	10.5	5.11	2.0 < = Vol < 5.0	2.40	0.85
5.0 < = Vol < 10.0	2.91	1.41	5.0 < = Vol < 10.0	0.19	0.07
10.0 < = Vol < 20.0	0.80	0.39	10.0 < = Vol < 20.0	0.02	0.01
20.0 < = Vol	0.16	0.08	20.0 < = Vol	0.01	0.00
Sum	206.4	100.0	Sum	283.5	100.0

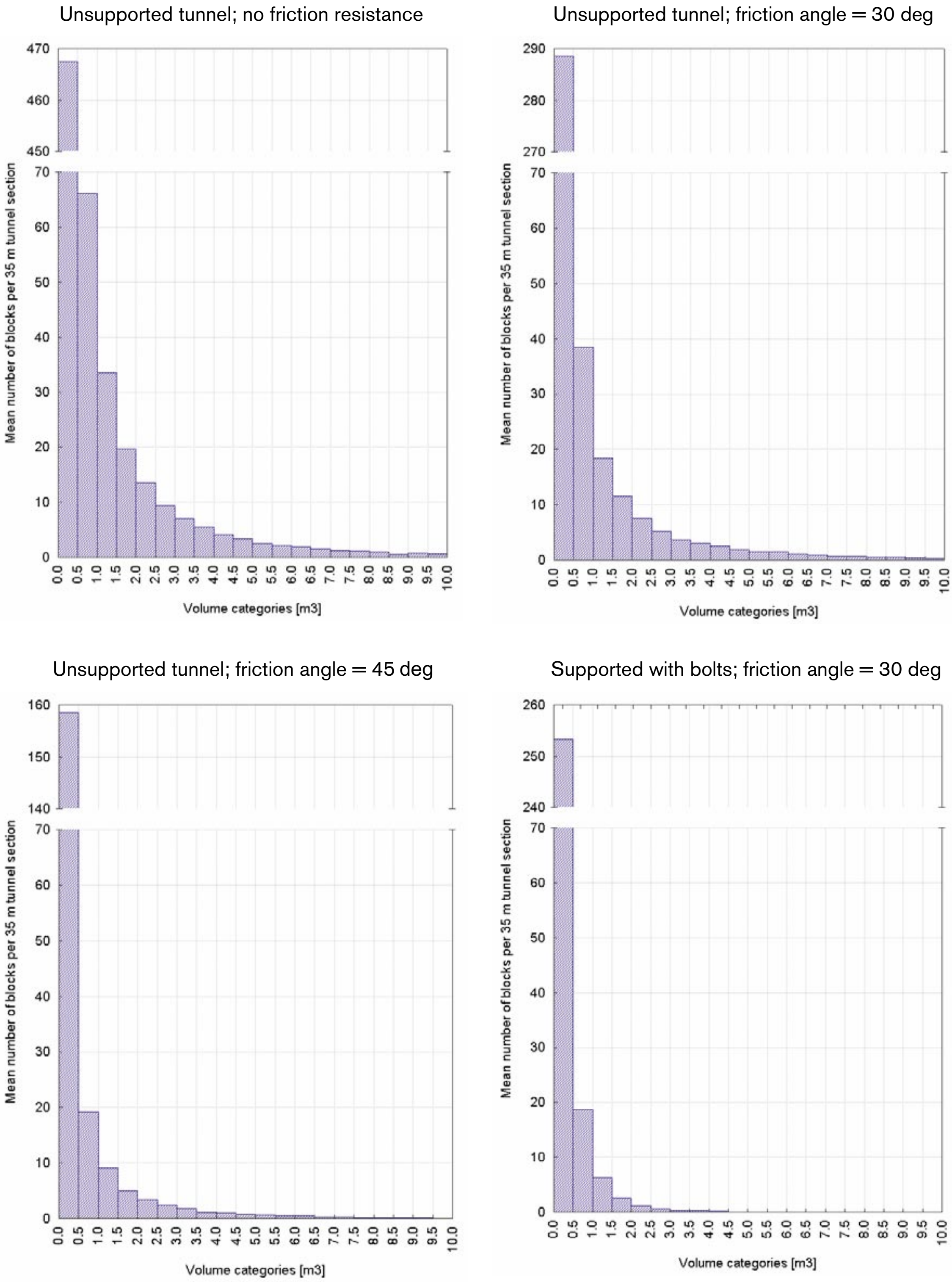

Figure A3-2. *Histograms of estimated unstable block volumes.*

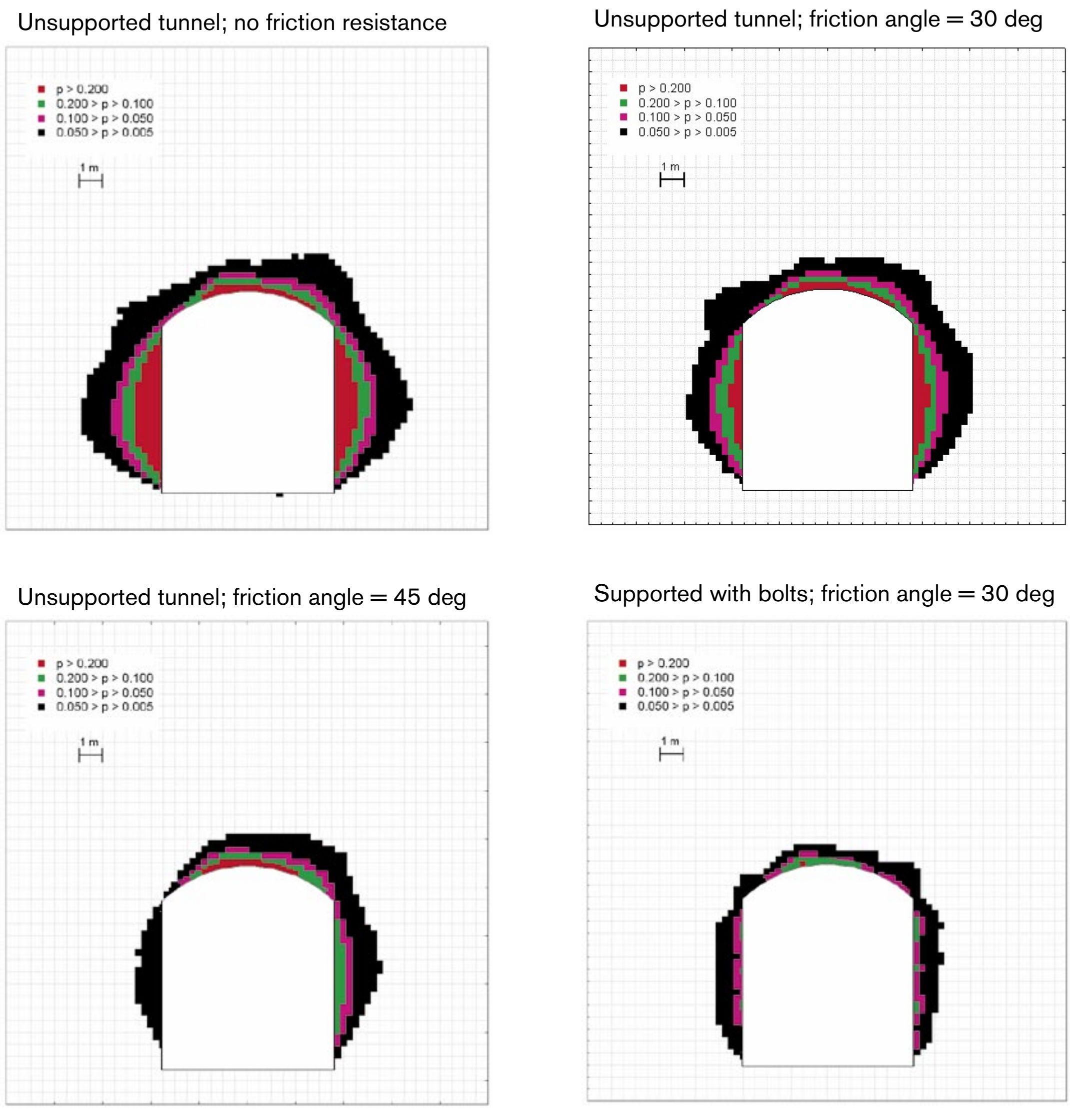

Figure A3-3. *Probability maps of unstable areas.*